ADDIS ABABA SCIENCE AND TECHNOLOGY UNIVERSITY

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GRADUATE PROGRAM

EVALUATION OF BRIDGE HYDRAULICS

(Case study on Alaba-Sodo-Arbaminch road bridges)

By

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A thesis submitted to College of Architecture and Civil engineering in partial fulfillment of requirement for Masters of Science in Hydraulics Engineering

Advisor

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Addis Ababa

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A Thesis Submitted in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Hydraulics Engineering of Addis Ababa science and Technology University.

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College: **College of Architecture and Civil Engineering**

Department: **Civil Engineering**

Program: **Hydraulics Engineering**

Major Advisor: **Mesay Daniel (Dr.)**

CERTIFICATION

I, the undersigned, certify that I read and hear by recommend for acceptance by Addis Ababa Science and Technology University a Thesis entitled "**Evaluation of Bridge Hydraulics''** ***a Case study on*** **"Alaba-Sodo-Arbaminch Road bridges'' '**in partial fulfillment of the requirement for the degree of Master of Science in Hydraulic Engineering.

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Dr. Mesay Daniel

Principal Advisor

DECLARATION AND COPY RIGHT

I, Mifta Sultan, declare that this Thesis is my own original work and that it has not been presented and will not be presented by me to any other University for similar or any other degree award.

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APPROVAL PAGE

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ABSTRACT

Hydraulic problem is one major reasons for bridge failure. In Ethiopia the hydraulic analysis and design of bridges is carried out using the procedure of drainage design manual of Ethiopian Roads Authority. This manual recommends demonstration of the design using computer models. However, the newly designed bridges pass through this step, the effectiveness of the hydraulic design procedure provided in the manual is not evaluated using the previously constructed bridges. This study intends to evaluate Bridge hydraulics with the aid of HEC-RAS and other supporting software. Five existing bridges were selected and examined. In addition, comparison between different bridge hydraulic analysis methods, transition lengths and boundary conditions were made. The main data used for this paper were rain fall data, Satellite Image and topographic maps, survey data, flow data and cross-section and geometric data for existing Structures. The results show Bishan Guracha and Wedeba River Bridge are hydraulically safe. But Raya River Bridge has small vertical clearance. In addition, overtopping of bridge is observed in the case of both Hammessa-1 and Baso River bridges. The study also found that serviceability of these bridges is questionable without planning permanent solutions.

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LIST OF ABBREVIATIONS

AEC	Associate Engineering Consult
BD and BU	Bridge upstream and Bridge downstream
BMS	Bridge Management System
CN	Curve Number
D/S	Downstream
DDM	Drainage Design Manual
DEM	Digital Elevation Model
ERA	Ethiopian Roads Authority
FHWA	Federal Highway Administration
GIS	Geographic Information System
HEC	Hydraulic Engineering Center
RAS	River Analysis System
SCS	Soil conservation service
TDOT	Tennessee Department of Transportation
U/S	Upstream
WSPRO	Water Surface Profile (model and bridge modeling approach in HEC-RAS)

1 INTRODUCTION

1.1 Background

The highway network of any country is one of the major public investments designed to support the national economy. When well developed and maintained, the road network is expected to meet the national objectives for road transport. The road assets in Ethiopia vary extremely from a limited number of 4 lane high speed highways to low volume community roads (ERA BMS Bulletin 2008).

Bridges are vital components of the road network and they form an essential part of the infrastructure of a nation, facilitating its social and economic development by allowing the free movement of people and goods between remote locations. Bridge is normally a man built structure that shall make it possible for traffic to cross one of nature or man built obstacle.

Now a time in our country there are lots of road projects which are under construction and lot of design works are being done for the future expansion of the road network throughout the country. Bridges are one of vital component of those road networks but the country has experienced many cases of bridge failure for many years. Therefore, designing of bridge must be exercised carefully.

Roadway Design, Hydraulic Design and Structural Design are three basic components of bridge designs. The main focus of this research is evaluating bridge hydraulics of Bishan Guracha, Wedeba, Hamassa-1, Raya and Baso River Bridges located on Alaba-Sodo-Arba Minch road.

The analysis were done using ArcGIS, HEC-HMS and Global Mapper software for the determination of the catchment characteristics and peak flood determination based on ERA (Ethiopian Roads Authority) Drainage Design Manual. And HEC–RAS 5.0, Civil 3D and HEC-Geo RAS with other data were used for the Hydraulic modelling. Based on the obtained finding’s mitigation measures were recommended.

1.2 Statement of the problem

Bridges are very essential components of road networks that contribute greatly to the national development and public daily life. Therefore any damage or failure of bridge will cause great negative impact on the life of road users and also require great amount of money to maintain or reconstruct the bridge.

Bridge failure can be defined as loss of a structural component, loss of a bridge's basic functionality, a catastrophic bridge collapse, or any damage condition in between. Bridge events including, flood and scour were the biggest causes of bridge problems in the USA. Almost 32.5 % of failure in USA is due to flood and 15.5% is due to scour. (McLinn, 2009)

Ethiopia also experienced many bridge failures. But no records are available to determine percent of bridge failures due to hydraulic problem. The bridge failures presented below are samples of different types of bridge failures observed in our country

- ✓ Delbena River Bridge located on the Arbaminch Konso road and it is failed due to flooding in Aug 2006 (Niguse, B.July 2010).
- ✓ Guang River Bridge is located on Metema Abederafi gravel road section around 8km from Metema. The bridge were constructed by nongovernmental organization (NGO) and the bridge serves only for one rainy season and completely failed due to flooding.
- ✓ The Fafem River Bridge is located on Harar Jijiga road project at 68km from the tourist town Harar. The bridge were constructed in 2007 G.C and failed within a year in 2008 G.C due to flooding.

Generally hydraulic problem is one of the major cause of bridge failures and several bridge failures occurred in Ethiopia. Therefore, evaluating bridge hydraulic become essential to identify status of bridges and to give awareness to government bodies to take a precautions. Besides, the findings from this thesis can be used for similar projects in the future.

1.3 Objectives

1.3.1 General objective

The main objective of this study is to evaluate bridge hydraulics in our country by the help of HEC-RAS hydraulic model and to contribute a better basis for Hydraulic design of bridges.

1.3.2 Specific objective

- ✓ To assess the effect of using different methods and recommendation during HEC-RAS modelling.
- ✓ To use the outcome result for further problem prediction and mitigation measures.
- ✓ To evaluate bridge hydraulics of Bihan Guracha, Wedaba, Raya, Baso and Hamassa-1 River Bridges

2 LITERATURE REVIEW

2.1 Literatures on Advancement of Bridge hydraulics

The evolution of hydraulic design illustrates the complexity of this facet of the bridge design process. The earliest methods for determining waterway openings for bridges and culverts did not consider bridge or culvert configuration. Furthermore, the concept of a "design" discharge or recurrence interval of expected floods to use when determining structure size was not considered.

(Byrne, 1893) Suggested that the factors to be considered when determining the capacity of a hydraulic culvert depended on;

- ✓ The rate of rainfall,
- ✓ The kind and condition of the soil
- ✓ The character and inclination of the surface
- ✓ The condition of inclination of the bed of the stream
- ✓ The shape of the area to be drained, and the branches of the stream
- ✓ The form of the mouth and the inclination of the bed of the culvert and
- ✓ Whether it is permissible to back the water up above the culvert, thereby causing it to discharge under a head.

To account for backwater, research was completed and methods were developed that examined the components of backwater (Liu, et al., 1957). In Hydraulic Design Series 1, the computed backwater was added to the "normal" depth at a location upstream of the bridge to evaluate the overall impacts of a bridge (Bradley, 1978).

Another significant development that contributed to the development of the current state of bridge hydraulics was the publication of a textbook open channel flow by V.T. Chow (Chow 1959). The publication presents and applies concepts of energy, momentum, and continuity to the flow of water in open channels. This literature also discussed direct and standard step methods for computing water surface profiles were first presented. The direct step method is applicable to prismatic channels and the standard step method to natural channels. The standard step method uses concepts of conservation of energy and flow, and is widely used for water surface profile calculations (Zevenbergen, et al., 2012).

In the late 1970’s and early 1980’s hydraulic engineers began to use computers to assist in their design work. The Corps of Engineers and the Federal Highways Administration (FHWA) both introduced programs for computing flood profiles through bridges.

In 1976 the Corps Hydraulic Engineering Center (HEC) introduced HEC-2 ‘Water Surface Profiles’ (HEC, 1982). This computer program was designed to compute water surface elevations along a stream or river reach. It was designed to accommodate bridges, culverts, dams, and weirs, as well as unconstricted reaches.

HEC-2 provided two methods for computing flow profiles through bridges:

- ✓ The normal bridge and
- ✓ Special bridge methods.

The normal bridge method computes a water surface profile through bridges by use of the energy equations and the standard step method. This method assumes energy losses are caused by flow contraction and expansion upstream and downstream of the bridge, and by friction. The special bridge method uses a method developed by Yarnell for factoring in the hydraulic effects of bridge piers. This empirical method was developed based upon over 2,100 flume experiments utilizing various shapes and sizes of bridge piers. Based upon these experiments, pier coefficients were developed to account for the most common shapes of bridge piers. This method requires only four cross-sections for computations. The bridge opening is approximated by a trapezoid.

HEC-2 was the first widely used computer program for hydraulic design of bridges. It has been used extensively in the National Flood Insurance Program for developing flood elevations, mapping floodplains, and designating floodplain widths to be used in the production of flood hazard maps.

In 1986, the Federal Highways Administration introduced a new methodology for hydraulic calculations at bridges and a computer program, Water Surface Profiles (WSPRO), based upon this methodology. WSPRO is similar to HEC-2, but while HEC-2 is intended for general flood profiles, WSPRO was specifically developed for bridge design applications.

WSPRO utilizes the standard step method for unconstricted sections. At bridge locations WSPRO uses special empirical methods for determining bridge losses. These methods were developed by the USGS for specific use in WSPRO and are somewhat different from the methods used by HEC-2 (Shearman, et al., 1986).

WSPRO proved very useful by automating part of the design process previously done by hand. However, it is not without its drawbacks. WSPRO and HEC-2 were developed originally for the punch cards used with early mainframe computers. With the advent of personal computers both were modified to use with personal computer operating systems. They utilize text only and are deficient in the area of graphical viewing of cross-sections and results. Debugging these can be daunting when faced with page after page crammed with text and numbers.

In the early 1990's computer software manufacturers introduced the concept of a graphical user interface. This type of interface represents files and objects as graphical icons. Introduction and popularization of the graphical user interface made it possible for software to use graphics extensively. As a consequence, software in general, and engineering software in particular, became much more user-friendly.

The Corps was quick to take advantage of this technological improvement. In 1995 HEC introduced the River Analysis System (HEC-RAS) (HEC, 1995). HEC's stated intention is for HEC-RAS to replace HEC-2. HEC-RAS provides capabilities similar to HEC-2. The major improvement however, is the addition of a graphical user interface. While requirements for data input by the user are similar between HEC-2 and HEC-RAS, the graphical capabilities of HEC-RAS provide great assistance in detecting bugs and errors in data input. Graphic capabilities for output data are much improved as well. Users can plot cross-sections and bridges and overlay water surface elevations as needed. This provides extensive help in visualizing situations and comparing alternatives. HEC-RAS also provides improved computation methods based upon new advances in hydraulic engineering theory since the introduction of HEC-2.

HEC-RAS has been part of the standard bridge design process at TDOT since late 1998. Reaction of TDOT engineers is mixed. HEC-RAS is much praised for its graphical capabilities; however developing a HEC-RAS bridge model is more time-consuming than with WSPRO.

Due to its increased flexibility and user-friendly graphics, HEC-RAS is becoming the method of choice for hydraulic bridge design. Based upon an informal survey conducted by the author of state Departments of Transportation in the southeast United States, WSPRO was the software of choice for the 1980's and early 1990's. The majority of state DOTs contacted is now using or considering the use of HEC-RAS for bridge designs. (Peck, 2001)

2.2 Hydraulic modeling criteria and selection

Any hydraulic model, whether it is numerical or physical, has assumptions and requirements. It is important for the hydraulic engineer to be aware of and understand the assumptions because they form the limitations of that approach. It is the goal of any hydraulic model study to accurately simulate the actual flow condition. Violating the assumptions and ignoring the limitations will result in a poor representation of the actual hydraulic condition. Treating the model as a black box will often produce inaccurate results. This is not acceptable given the cost of bridges and the potential consequences of failure. Therefore, the approach should be selected based primarily on its advantages and limitations, though also considering the importance of the structure, potential project impacts, cost, and schedule. (Zevenbergen, et al., 2012)

Flow through bridges can be simulated as either one-dimensional or two-dimensional flow. Bridge flow probably needs to be modeled with two-dimensional elements if the width of the bridge is large in comparison to the width of the channel or floodplain on which it is located.

Although the representations of the variables may differ between the various two-dimensional modeling programs, and some variables may not be included in all the programs, the conservation of mass and momentum are used as the basis for hydraulic calculation in two-dimensional models.

There are definitely some areas where 2D modeling can produce better results than 1D modeling, and there are also situations in which 1D modeling can produce just as good of results as or better than 2D model with less effort and computational requirements.

Unfortunately, there is a very large range of situations that fall into a gray area, and one could list the positive and negative aspects of both methodologies for specific applications.

Table 2-1 comparison between one dimensional and two dimensional model

Bridge Hydraulic Modeling Selection.		
Bridge Hydraulic Condition	*Hydraulic Analysis Method*	
	One-Dimensional	*Two-Dimensional*
Small streams	●	◗
In-channel flows	●	◗
Narrow to moderate-width floodplains	●	◗
Wide floodplains	◗	●
Minor floodplain constriction	●	◗
Highly variable floodplain roughness	◗	●
Highly sinuous channels	◗	●
Multiple embankment openings	◗/○	●
Unmatched multiple openings in series	◗/○	●
Low skew roadway alignment (<20˚)	●	◗
Moderately skewed roadway alignment (>20˚ and <30˚)	◗	●
Highly skewed roadway alignment (>30˚)	○	●
Detailed analysis of bends, confluences and angle of	○	●
Multiple channels	◗	●
Small tidal streams and rivers	●	◗
Large tidal waterways and wind-influenced conditions	○	●
Detailed flow distribution at bridges	◗	●
Significant roadway overtopping	◗	●
Upstream controls	○	●

Countermeasure design	◗	●
● *well suited or primary use* ◗ *possible application or secondary use* ○ *unsuitable or rarely used* ◗/○ *possibly unsuitable depending on application*		

For this thesis HEC-RAS Hydraulic model is selected. Extensive documentation concerning HEC-RAS is available from HEC. HEC provides a detailed discussion of the theory of RAS in its manual. It also contains recommendations for dealing with various modeling situations the user may encounter. Further details and discussion can be found within the course notes provided as part of HEC-RAS training classes offered by HEC and the National Highways Institute.

While HEC (1997) provides an overview of HEC-RAS's application of the WSPRO method, (Shearman, et al., 1986)discuss the WSPRO methodology in detail as it was originally implemented. Shearman provides theoretical background and data requirements for using this method for bridge analysis.

Brunner and Hunt (1995) performed a comparison of HEC-RAS, WSPRO, and HEC-2. Their study contains a discussion of the similarities and differences of the fundamental computational methods of each and how often cross-sections should be placed. Using a sample consisting of thirteen bridge sites located in Louisiana, Alabama, and Mississippi with seventeen flood flows they determined the mean average absolute error for each computation method by comparing calculated water surface elevations to observed field data. Based on these results they concluded that all three programs computed water surface elevations "within the tolerances of observed data" and in their comparison of modeling software types Brunner and Hunt (1995) find location of cross- sections to be more important than the type of model used, however, they do not provide guidance concerning this.

2.3 HEC-RAS Hydraulic Model

2.3.1 Basic hydraulic theory of HEC-RAS

Any hydraulic model, whether it is numerical or physical, has assumptions and requirements. These assumptions generally simplify calculations by eliminating factors which do not significantly affect the outcome. It is important for the hydraulic engineer to be aware of and understand the assumptions because they form the limitations of that approach. This requires a good knowledge of hydraulic theory and its application to the situation being modeled. Simplifying assumptions may also limit the situations in which a mathematical model may be used with validity. When using a mathematical model the user must understand the limiting assumptions and their effect upon the model results. This is a key factor when choosing which available modeling software to use for various applications. The user must understand the limiting assumptions in order to correctly apply the modeling software and interpret its results.

This work use HEC-RAS numerical model to evaluate hydraulic design practice of bridge in our country. Assumptions are made during the development of those models. A discussion of the basic theory and limiting assumptions will provide back ground for the reader's benefit. But this discussion is not comprehensive, and the reader should refer to the HEC-RAS reference manual and user guide for more detailed information from their website.

Basic hydraulic Theories which are used by HEC-RAS hydraulic model are

- ✓ Open Channel Flow Theory
- ✓ Normal Depth
- ✓ The Standard Step Method
- ✓ Critical, Sub-critical and super critical flow
- ✓ Momentum Equation

2.3.2 Basic data requirement

The data needed to perform these computations are divided into the following categories: geometric data; steady flow data; unsteady flow data; and sediment data. Geometric data are required for any of the analyses performed within HEC-RAS. The other data types are only required if you are going to do that specific type of analysis (i.e., steady flow data are required to perform a steady flow water surface profile computation). The current version of HEC-RAS can perform either steady or unsteady flow computations.

- **Geometric Data:**

The basic geometric data consist of establishing the connectivity of the river system (River System Schematic); cross section data; reach lengths; energy loss coefficients (friction losses, contraction and expansion losses); and stream junction information. Hydraulic structure data (bridges, culverts, spillways, weirs, etc...), which are also considered geometric data, will be described in later chapters

- **River system schematic**

The schematic defines how the various river reaches are connected. The program can handle simple single reach modules or complex networks. The river system schematic is developed by drawing and connecting the various reaches of the system within the geometric data editor

- **Cross section Geometry**

Boundary geometry for the analysis of flow in natural streams is specified in terms of ground surface profiles (cross sections) and the measured distances between them (reach lengths). Cross sections should be perpendicular to the anticipated flow lines and extend across the entire flood plain (these cross sections may be curved or bent).

Cross sections require at locations where changes occur in discharge, slope, shape or roughness; at locations where levees begin or end and at bridges or control structures such as weirs.

- **Reach length**

The measured distances b/n cross sections are referred to as reach lengths. The reach length (distance between cross sections) should be measured along the anticipated path of the center of mass of the left and right over bank and the center of the channel (these distances may be curved).

- **Energy loss coefficients**

Several types of loss coefficients are utilized by the program to evaluate energy losses: (1) Manning's n values or equivalent roughness "k" values for friction loss, (2) contraction and expansion coefficients to evaluate transition (shock) losses, and (3) bridge and culvert loss coefficients to evaluate losses related to weir shape, pier configuration, pressure flow, and entrance and exit conditions

Manning's n.

Selection of an appropriate value for Manning's n is very significant to the accuracy of the computed water surface profiles. The value of Manning's n is highly variable and depends on a number of factors including: surface roughness; vegetation; channel irregularities; channel alignment; scour and deposition; obstructions; size and shape of the channel; stage and discharge; seasonal changes; temperature; and suspended material and bed load. Three values of n will be selected for each cross section; n for the left and right overbank and n for the center of the channel.

Contraction and expansion coefficients

Contraction or expansion of flow due to changes in the cross section is a cause of energy loss between cross sections. The loss may be computed from the contraction and expansion coefficients specified on the cross section data editor. The coefficients, which are applied between cross sections, are specified as part of the data for the upstream cross section. The coefficients are multiplied by the absolute difference in velocity heads between the current cross section and the next cross section downstream, which gives the energy loss caused by the transition.

- **Steady Flow Data**

Steady flow data are required in order to perform a steady water surface profile calculation. Steady flow data consist of: flow regime; boundary conditions; and peak discharge information.

- **Unsteady Flow Data**

Unsteady flow data are required in order to perform an unsteady flow analysis. Unsteady flow data consists of boundary conditions (external and internal), as well as initial conditions.

2.3.3 HEC-RAS Bridge Cross-sections

The bridge routines utilize four user-defined cross sections in the computations of energy losses due to the structure. During the hydraulic computations, the program automatically formulates two additional cross sections inside of the bridge structure. The Figure below shows recommended cross-section locations. These sections referred to as the bridge reach.

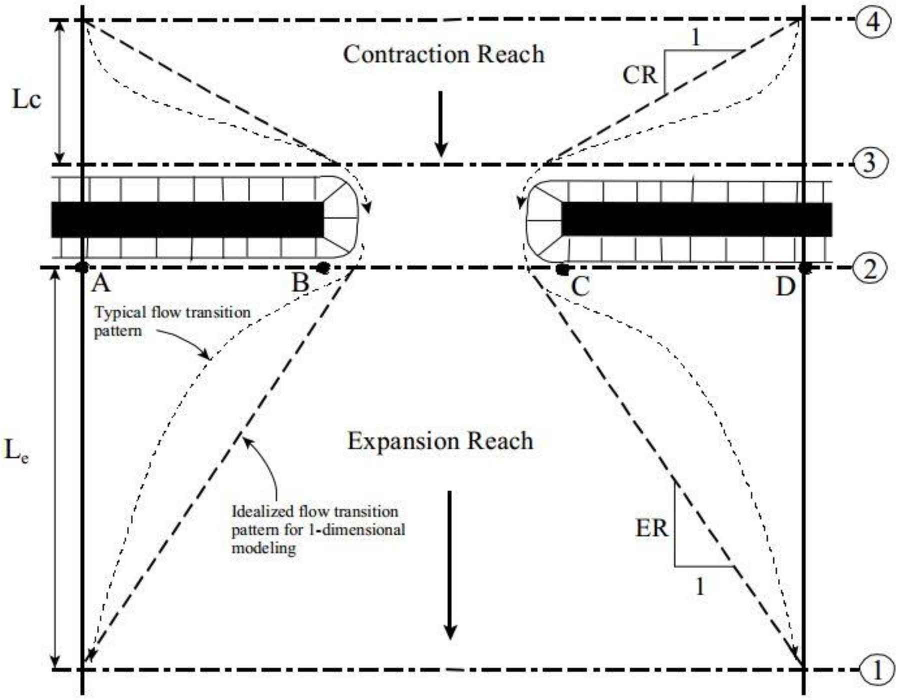

Figure 2-1: Bridge Reach Cross-Sections

Cross section 1 is located sufficiently downstream from the structure so that the flow is not affected by the structure (i.e., the flow has fully expanded). Cross section 4 is an upstream cross section where the flow lines are approximately parallel and the cross section is fully effective. In general, flow contractions occur over a shorter distance than flow expansions. Cross-section 1 commonly referred to as the exit section. Whereas, cross-section 4 is referred to as the approach section. These names are due to the fact that flow is approaching or exiting the bridge reach at each section.

Cross section 2 is located a short distance downstream from the bridge (i.e., commonly placed at the downstream toe of the road embankment). This cross section should represent the natural ground (main channel and floodplain) just downstream of the bridge or culvert. This section is normally located near the toe of the downstream road embankment. This cross section should not be placed immediately downstream of the face of the bridge deck or the culvert opening.

Cross section 3 should be located a short distance upstream from the bridge (commonly placed at the upstream toe of the road embankment). The distance between cross section 3 and the bridge should only reflect the length required for the abrupt acceleration and contraction of the flow that occurs in the immediate area of the opening. Cross section 3 represents the natural ground of the channel and overbank area just upstream of the road embankment. This section is normally located near the toe of the upstream road embankment. This cross section should not be placed immediately upstream of the bridge deck. The bridge routines used between cross sections 2 and 3 account for the contraction losses that occur just upstream of the structure (entrance losses). Therefore, this cross section should be place just upstream of the area where the abrupt contraction of flow occurs to get into the bridge opening. This distance will vary with the size of the bridge opening.

Both cross sections 2 and 3 will have ineffective flow areas to either side of the bridge opening during low flow and pressure flow. In order to model only the effective flow areas at these two sections, the modeler should use the ineffective flow area option. This option is selected from the cross section data editor. Ineffective flow areas do not conduct flow, but do provide flood storage. These areas may become effective if the water surface increases above a user specified elevation.

2.3.4 HEC-RAS high and low flow bridge computations

Free-surface bridge flow (low flow) refers to the range of flow conditions at a specific bridge in which the bridge low chord is not submerged. The HEC-RAS Hydraulic Reference Manual classifies free surface bridge flow conditions as Class A, Class B or Class C, depending on flow regime in the stream reach being crossed and in the bridge waterway itself.

Overtopping flow is the condition in which flow is crossing over the roadway approaches or the bridge deck itself. Overtopping flow conditions are appropriately represented by a broad- crested weir. Overtopping flow at bridge crossings is usually combined with either free-surface bridge flow or submerged-deck flow in the bridge waterway. When overtopping flow occurs, the engineer must determine how much flow is going through the bridge and how much over the bridge deck or roadway.

A condition in which the water surface is above the highest point of the bridge low chord is usually representative of orifice flow. When the low chord is submerged only at the upstream edge of the superstructure, the orifice is considered free-flowing, and thus not affected by tail water. This condition is analyzed using the same approach as for an orifice. Another type of orifice flow exists when the highest point of the low chord is submerged at both the upstream and downstream edges of the superstructure. This type of flow is analyzed using a formulation for a tail water-controlled orifice (Bradley, 1978)

The bridge routines in HEC-RAS allow the modeler to analyze a bridge with several different methods without changing the bridge geometry. The bridge routines have the ability to model low flow (Class A, B, and C), low flow and weir flow (with adjustments for submergence on the weir), pressure flow (orifice and sluice gate equations), pressure and weir flow, and highly submerged flows (the program will automatically switch to the energy equation when the flow over the road is highly submerged).

- **HEC-RAS Low Flow Bridge Computations**

Low flow exists when the flow going through the bridge opening is open channel flow. Class A is the most commonly encountered free-surface bridge flow condition. In this class of flow the conditions are subcritical upstream of the bridge, downstream of the bridge, and throughout the bridge waterway. Class A flow generally satisfies the constraints of gradually varied flow throughout the reach of interest. HEC-RAS provides four available approaches to modeling Class A free-surface bridge flow at a bridge. These four methods for computing losses through the bridge are: Energy Equation (standard step method), Momentum Balance, Yarn ell Equation and FHWA WSPRO method. The user can select any or all of these methods to be computed.

Energy Equation (standard step method)

The energy-based method treats a bridge in the same manner as a natural river cross-section as discussed previously, except the area of the bridge below the water surface is subtracted from the total area, and the wetted perimeter is increased where the water is in contact with the bridge structure. This method does not account for pier drag losses or pier and abutment shapes.

The program formulates two cross sections inside the bridge by combining the ground information of sections 2 and 3 with the bridge geometry as shown in Figure below. The sequence of calculations starts with a standard step calculation from just downstream of the bridge (section 2) to just inside of the bridge (section BD) at the downstream end. The program then performs a standard step through the bridge (from section BD to section BU). The last calculation is to step out of the bridge (from section BU to section 3).

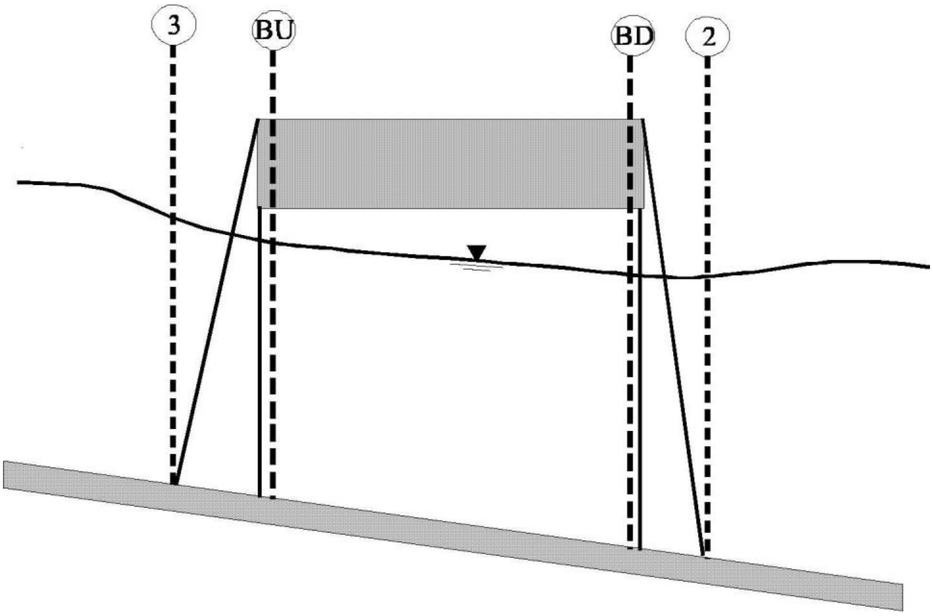

Figure 2-2: Cross Sections Near and Inside the Bridge

Momentum Balance Method

The momentum method is based on performing a momentum balance from cross section 2 to cross-section 3 by using momentum equation discussed above. The momentum balance is performed in three steps. The first step is to perform a momentum balance from cross section 2 to cross-section BD inside the bridge. The equation for this step is as follows:

$$A_{BD}\bar{Y}_{BD} + \frac{\beta_{BD}Q_{BD}^2}{gA_{BD}} = A_2\bar{Y}_2 + \frac{\beta_2 Q_2^2}{gA_2} - A_{PBD}\bar{Y}_{PBD} + F_f - W_x \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.1$$

The second step is a momentum balance from section BD to BU (see Figure above). The equation for this step is as follows:

$$A_{BU}\overline{Y}_{BU} + \frac{\beta_{BU}Q_{BU}^{2}}{gA_{BU}} = A_{BD}\overline{Y}_{BD} + \frac{\beta_{BD}Q_{BD}^{2}}{gA_{BD}} + F_f - W_x \dots\dots\dots\dots\dots\dots\dots\dots 2.2$$

The final step is a momentum balance from section BU to section 3 (see Figure above). The equation for this step is as follows:

$$A_3\overline{Y}_3 + \frac{\beta_3 Q_3^{2}}{gA_3} = A_{BU}\overline{Y}_{BU} + \frac{\beta_{BU}Q_{BU}^{2}}{gA_{BU}} + A_{PBU}\overline{Y}_{PBU} + \frac{1}{2}C_D\frac{A_{PBU}Q_3^{2}}{gA_3{}^{2}} + F_f - W_x \dots\dots 2.3$$

Where In the equations above:Ai = Active flow area at the cross section denoted by the subscript, ft2 (m2)

$A_{P\,BU}$, $A_{P\,BD}$ = Flow area obstructed by pier at the upstream and downstream faces of the bridge opening, ft^2 (m^2)

Yi = Vertical distance from the water surface to the centroid of the flow area at the cross section denoted by the subscript, ft (m)

Y_{PBU}, Y_{PBD} = Vertical distance from water surface to the centroid of the pier area at the upstream and downstream faces of the bridge opening, ft (m)

Qi = Discharge at the cross section denoted by the subscript, ft3/s (m3/s)

ßi = Velocity weighting coefficient for momentum at the cross section denoted by the subscript

Ff = External friction force acting on the control volume per unit weight of water, ft3 (m3)

Wx = Component of the weight of water acting in the direction of flow, per unit weight of water, ft3 (m3)

CD = Drag coefficient for flow around the pier

Drag coefficients are used to estimate the force due to the water moving around the piers, the separation of the flow, and the resulting wake that occurs downstream. The user enters the drag coefficient, which is a function of the plan-view shape of the pier. Recommended values for this coefficient range from 0.29 for elliptical piers to 2.00 for square nose piers. Because of the pier drag coefficient, the Momentum Balance Method is sensitive to the hydraulic efficiency of the pier shape. This is an advantage over the Energy Method, which does not provide a way of accounting for streamlined pier shapes. The Momentum Balance Method is also the preferred approach to computing the bridge hydraulics in Class B flow, because it is not hindered by rapidly varied flow conditions.

Yarn ell Equation.

While the Energy Method and the Momentum Balance Method are theoretically derived, the Yarn ell Equation is strictly empirical. It is based on the results of roughly 2600 flume experiments that were designed to test the relationship between the change in water surface elevation caused by a pier and the size, shape, and configuration of the pier in combination with varied flow rates. The resulting equation is:

$$H_{3-2} = 2K(K + 10\omega - 0.6)(\alpha + 15\alpha^4)\frac{V^2}{2g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.4$$

Where:

H_{3-2} = Drop in water surface elevation from the upstream bounding section

K = Yarn ell's pier shape coefficient (see below)

ω = Ratio of the velocity head to the depth at the downstream bounding section

Yarn ell's equation is especially sensitive to the pier shape coefficient, K, which varies from 0.90 for piers with semi-circular nose and tail to as much as 2.50 for ten pile trestle bents. A disadvantage of the Yarn ell Equation is that, because it is strictly empirical, its application should be limited to bridge sites that are similar in nature to the flume studies that were used in the development of the equation.

WSPRO Method

HEC-RAS also used WSPRO Method, which was adapted from the WSPRO computer program. The WSPRO Method is based on a standard-step solution of the energy equation, and is similar to the Energy Method in most respects. Unlike the other three free-surface bridge flow methods discussed here, the WSPRO Method works from the exit section to the approach section, and not just between the upstream and downstream bounding sections. A general energy balance equation from the exit section to the approach section can be written as follows:

$$h_4 + \frac{\alpha_4 V_4}{2g} = h_1 + \frac{\alpha_1 V_1}{2g} + h_{L(4-1)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.5$$

Where: h = Water surface elevation at section

V = Velocity at section

$h_{L(4-1)}$ = Energy losses from section 4 to 1

The incremental energy losses from section 4 to 1 are calculated as follows:

- ✓ From Section 1 to 2

Losses from section 1 to section 2 are based on friction losses and an expansion loss. Total losses between sections 1 and 2 are a combination of friction losse (equqtion below) and expansion loss (eq below).

$$h_{f(1-2)} = \frac{BQ^2}{K_2 K_1} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots.\dots 2.6a$$

Where: hf (1-2) = Total friction losses (m).

B = Flow distance (m).

Q = Flow (m3/sec).

K2, K1 = Conveyance at sections 1 and 2

$$h_e = \frac{Q^2}{2gA_1^2}\left[2\beta_1 - \alpha_1 - 2\beta_1\left(\frac{A_1}{A_2}\right) + \alpha_2\left(\frac{A_1}{A_2}\right)^2\right] \dots\dots\dots\dots\dots\dots.\dots 2.6b$$

Where: A = Flow area (m^2).

α, β *=Momentum correction factors for nonuniform flow.*

The momentum correction factors are calculated as a function of conveyance and area in open channel sections. However, WSPRO utilizes a special method for relating these correction factors to bridge geometry. An empirical coefficient of discharge, C, is used as shown below.

$$\alpha_1 = \frac{1}{C^2}$$

$$\beta_1 = \frac{1}{C}$$

- ✓ From Section 2 to 3

Losses from section 2 to section 3 are based on friction losses only. The energy balance is performed in three steps. From section two to BD, from BD to BU, and from BU to section three. Equation 25 shows the computation as applied between BD and BU. and Similar equations are used for the friction losses from section 2 to BD and BU to 3

$$h_{f(BU-BD)} = \frac{L_B Q^2}{K_{BU} K_{BD}} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots.\dots 2.6c$$

Where: KBU, KBD = Conveyance at respective sections.

LB = Length between sections (m).

Q = Flow (m3/sec).

- ✓ From Section 3 to 4

Energy losses from section 3 to 4 are based on friction losses only. The equation for computing the friction loss is as follows:

$$h_{f(3-4)} = \frac{L_{av}Q^2}{K_3K_4} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.\ldots 2.6d$$

Where Lav = is the effective flow length in the approach reach

K3 and K4 = the total conveyances at sections 3 and 4.

The effective flow length is computed as the average length of 20 equal conveyance stream tubes (Shearman, et al., 1986). The computation of the effective flow length by the stream tube method is explained by Appendix D of the HEC-RAS Hydraulic Reference Manual (Brunner, 2010) in detail.

- **HEC-RAS High Flow Bridge Computations**

High flow occurs when flow comes into contact with the maximum low chord of a bridge deck. There are two separate types of flow that may occur: pressure flow, and weir flow. These may occur separately or together, or weir flow may occur along with low flow.

Pressure flow begins when the water surface comes into contact with the upstream low chord of the bridge and Weir flow begins when the water surface rises above the lowest point of the embankment of the approach roadway. The roadway surface acts as a weir conducting water across the embankment and downstream of the bridge.

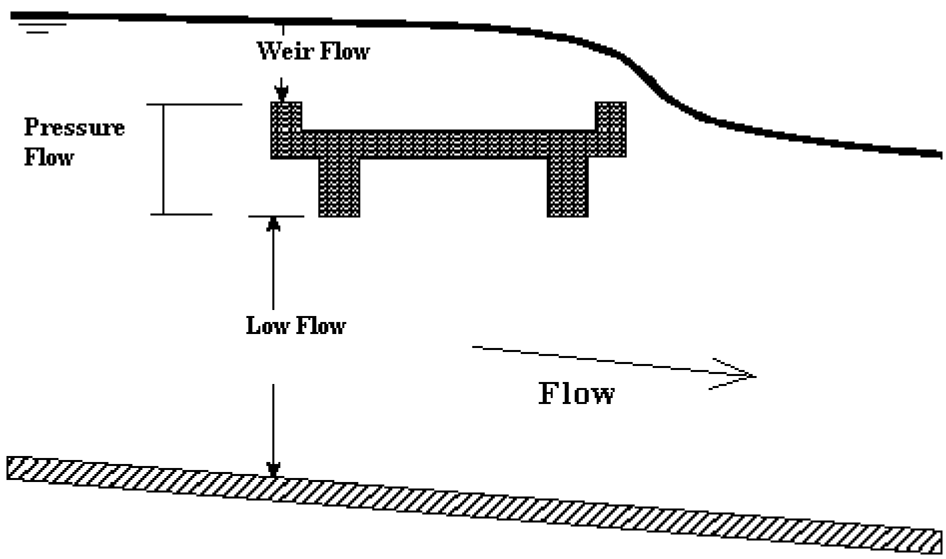

Figure 2-3: Low Flow, Pressure Flow, and Weir Flow Through A Bridge Opening

The HEC-RAS program makes two different approaches available to the user for modeling high flow conditions: by either the Energy equation (standard step method) or by using separate hydraulic equations for pressure and/or weir flow

Energy Method

Just as described above for free-surface bridge flow modeling, the Energy Method simply continues the standard-step solution of the energy equation through the bridge structure and vicinity. It accounts for the blockage caused by the road embankments, abutments, bridge deck and piers simply by reducing the conveyance. Computations are based on balancing the energy equation in three steps through the bridge. Energy losses are based on friction and contraction & expansion losses. Output from this method is available at the cross sections inside the bridge as well as outside.

If the water surface is high enough to overtop the road, the program will treat the flow area above the road as conveyance area, but not as a weir. When the Energy Method is used, the quantity of overtopping flow will not be computed or reported. If the low chord is submerged, the added wetted perimeter will have a negative effect on conveyance, but the program will not attempt to compute orifice conditions.

Pressure and Weir Flow Method

A second approach for the computation of high flows is to utilize separate hydraulic equations to compute the flow as pressure and/or weir flow.

Pressure flow occurs through the bridge opening when the water surface raises above the bridge low chord elevation. If water contacts only the upstream side then the equation for flow through a sluice gate is used (below).

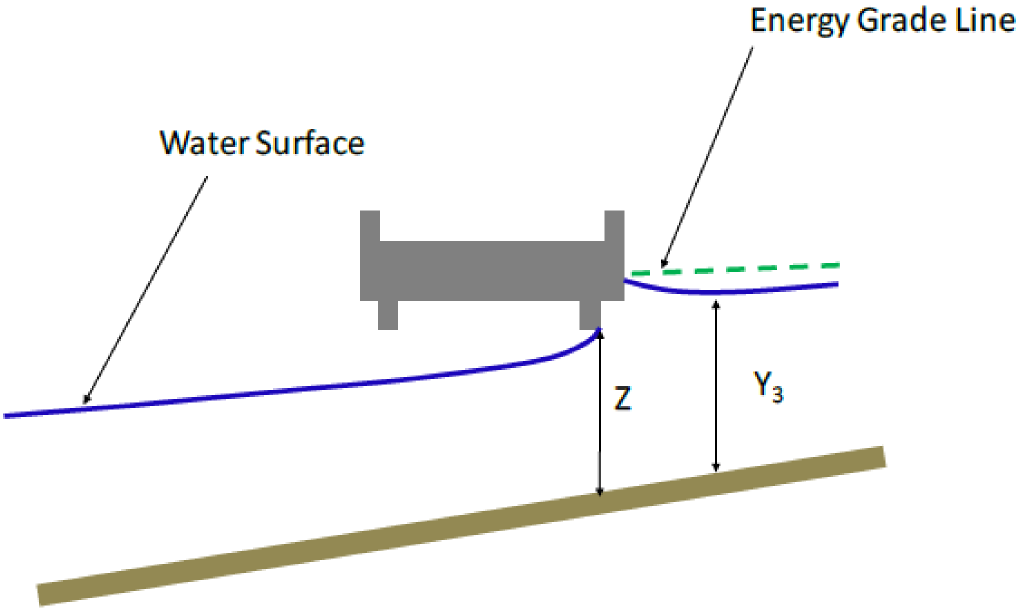

Figure 2-4: Sluice gate flow

$$Q = C_d A_{BU} \left[2g \left(Y_3 - \frac{Z}{2} + \frac{\propto_3 V_3^2}{2g} \right) \right]^{1/2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.7$$

Where: Q = Flow through bridge (m^3/sec).

Cd= Coefficient of discharge for pressure flow.

A_{BU} = Net area of bridge opening at section BU (m3)

Y3 = Hydraulic depth at section 3 (m).

Z = Vertical distance from max bridge low chord to mean river bed elevation at section BU (m)

If the water surface contacts both the upstream and downstream low chords then it is assumed the bridge opening is flowing full and the equation for flow through an orifice is used (27).

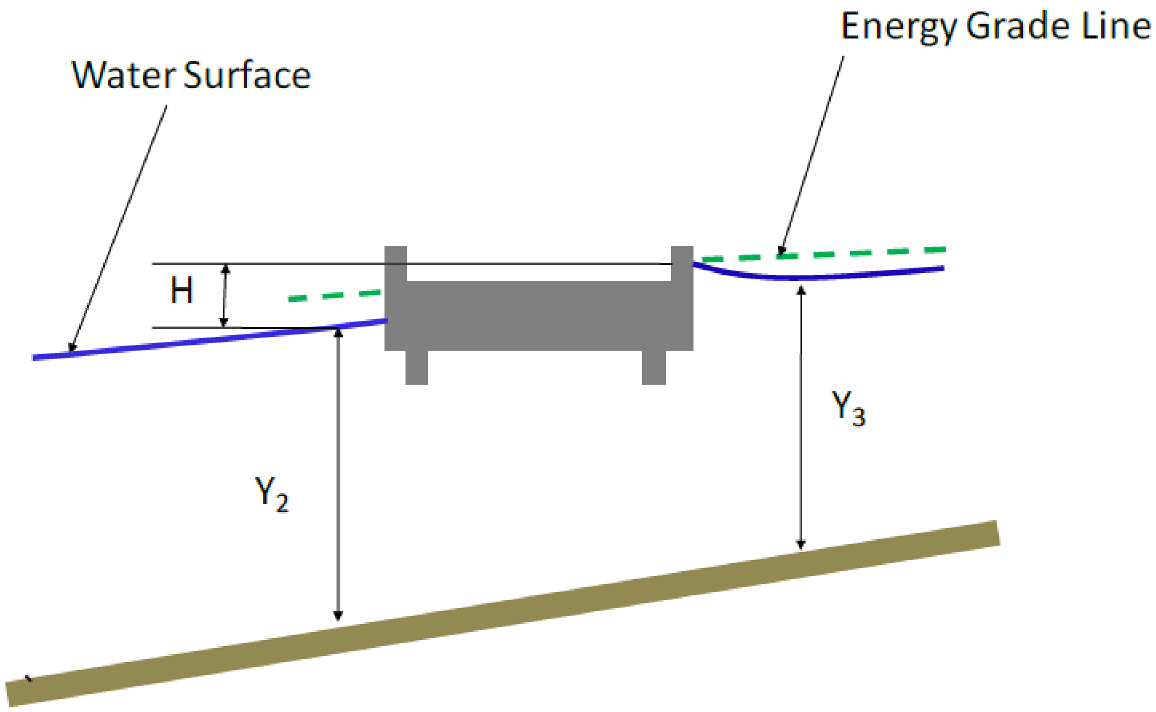

Figure 2-5: Orifice flow

$$Q = CA\sqrt{2gH} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.8$$

Where : Q = Flow through bridge (m3/sec).

C = Coefficient of discharge for fully submerged pressure flow.

H = Difference between the energy gradient elevation upstream of the bridge and the water surface elevation downstream of the bridge (m).

A = Net area of bridge opening (m2).

3 METHODOLOGY

3.1 Descriptions of Study Area

Arabaminch is a city and separate Woreda in southern Ethiopia located in the Gamo gofa Zone of the Southern Nations, Nationalities, and Peoples Region about 500 kilometers south of Addis Ababa, at an elevation of 1285 meters above sea level.

The study areas are located in Alaba - Sodo - Arabaminch road. The road serves as one of the major links to the South-Western Ethiopia

The road starts at Alaba town located 309kms south from Addis Ababa via Zeway and Shashamene. The project road proceeds in the south direction from Alaba through Sodo to Arbaminch. The length of the road is 185.6 km.

The existing road is double surfaced road which has been routinely maintained by ERA's Sodo District. The maintenance activities consisted of pavement patching works, pothole repair, crossing structure cleaning, river sedimentation dredging, ditch cleaning, shoulder balding etc.

The bridges used for this study are Bishan Guracha river bridge, Wedeba River Bridge, Raya River Bridge, Hamessa1 River Bridge and Baso River Bridge. Information of bridges are summarized in table 3.1

Table 3-1 Bridges information

Bridge No.	***River Name***	***Station***	***Km from Addis***	***X-Cord***	***Y-Cord***	***Span (m) & Arrangement***	***Bridge Type***
1	*Bishan Guracha*	*2+053*	*320.4*	*396931*	*803633*	*13*	*Single span deck-girder bridge.*
2	*Wedeba*	*35+633*	*354*	*381513*	*779106*	*9*	*Single span slab bridge.*
3	*Hamessa 1*	*92+414*	*410.4*	*369305*	*734966*	*14+9*	*Double span deck-girder and slab bridge.*
4	*Raya*	*117+805*	*436.2*	*361236*	*714043*	*45.8*	*Continuous Span RC deck-girder bridge.*
5	*Baso*	*161+613*	*480.3*	*352016*	*679052*	*17+17+17*	*Triple span deck-girder bridge.*

Figure 3-1 Study areas

3.2 Data collection and Modeling

3.2.1 General

Bridge sites were selected for inclusion in this study based upon available data. All sites were located on Alaba-Sodo-Arbaminch road. In general, all stream slopes were mild in all cases, causing low velocity conditions. Three sites were experienced low flow and two site experienced weir flow conditions

Available sources of data include; the bridge survey, Satellite Image and maps, couture map, rainfall data, flow data and cross-section and geometric data for existing structures. Rain fall data as secondary data from ACE consulting, flow data from Ministry of Water Resource and topographic map which include Digital Elevation Model (DEM) data of the project area on 30mx30m Grid from Ethiopian Mapping Agency were used for hydrologic analysis. Couture map, topographic map, bridge survey and geometric data for existing structures were used for hydraulic analysis.

3.2.2 River System Schematic

The main data used to create River system schematic was couture map. 1m interval couture map was obtained from ACE (Associate consulting Engineers). River system schematic defines how the various river reaches are connected. It is developed by drawing from upstream to downstream and connecting the various reaches of the system within the geometric data editor.

For this thesis river system schematics are imported from HEC-Geo RAS among with some other data (See the fig 3.2). The process is summarized below.

- ✓ Preparing contour map for specified area
- ✓ Creating surface profile using Auto CAD Civil-3D
- ✓ Export surface profile as DEM to Global Mapper
- ✓ To make DEM compatible with Arc GIS export DEM from Global Mapper to GIS
- ✓ Prepare the DEM data using HEC-Geo RAS (Arc GIS extension)
- ✓ Export from HEC-Geo RAS

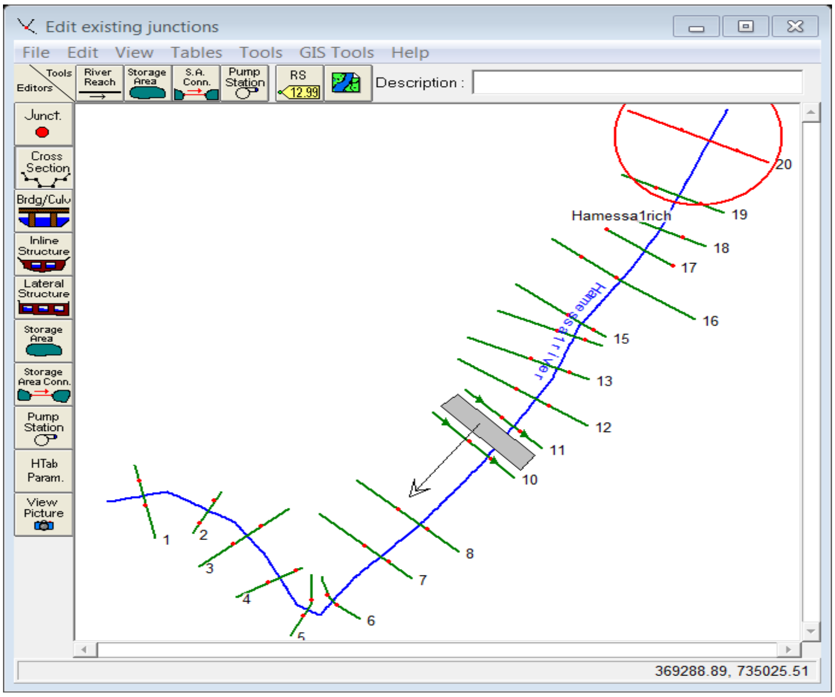

Figure 3-2: River System Schematic for Hamessa-1 River (HEC-RAS software)

3.2.3 Cross Section Geometric Data

In HEC-RAS the cross section geometric data consists of the: X-Y coordinates, reach lengths, bank station, Manning's n values and contraction & expansion coefficients. Data used to enter Cross Section Geometric were field survey data and topographical map.

Topographic maps, obtained from Mapping Agency, used to determine the geographic layout of the site and can provide information on adjacent properties that may be subjected to increased risk of flood damage by the proposed structure. Field survey was conducted by ACE (Associate consulting Engineers), which consists the following data

- ✓ River cross section in upstream and downstream of the bridge location.
- ✓ The river profile in upstream and downstream of the bridge
- ✓ Measurement of the bridge Length, Width, recent height i.e. above silt and height above excavated silt.
- ✓ The Asphalt Road profile 30m upstream and downstream of the bridge.

Fig 3.3 show HEC-RAS cross-section data editor with all the required data fields.

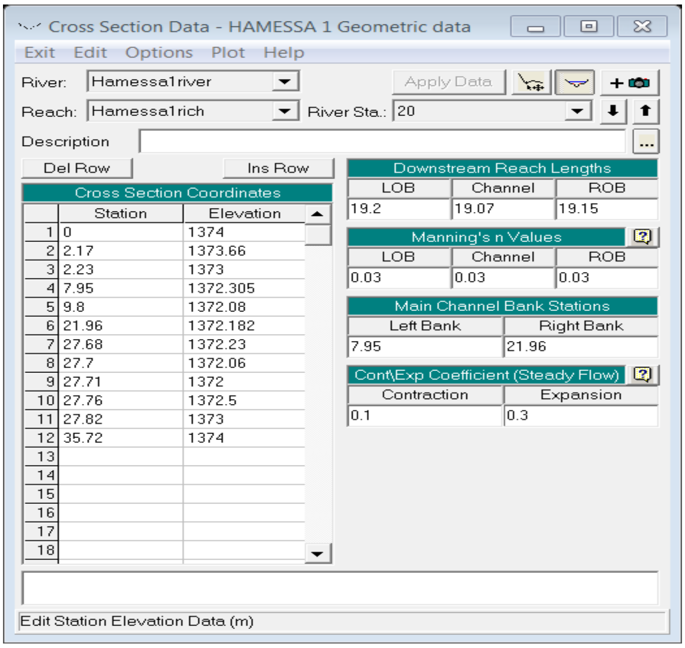

Figure 3-3: Cross-section data editor (HEC-RAS software)

X-Y coordinates used to represent the geometry of cross section. After cross sectional data imported from HEC-Geo RAS with river schematic, each Cross-sections adjusted manually according to field survey. HEC-RAS requires the user to furnish a minimum of four channel cross-sections in order to properly represent a bridge. It is highly advisable to provide other cross-sections outside the influence of the bridge. These sections help to include conditions that influence the water surface elevations at the bridge and ensure accurate results. Cross-section data is entered into HEC-RAS as a series of stations and corresponding elevations.

The distances between the cross sections are entered as the downstream reach lengths in the Cross Section Data Editor. The reach lengths determine the placement of the cross sections. The placement of the cross sections relative to the location of the bridge is crucial for accurate prediction of expansion and contraction losses. The bridge routine utilizes four cross sections to determine the energy losses through the bridge. Those cross- section refers to the section at which computations begin. This is the most downstream cross-section for sub-critical flow and the most upstream cross-section for super-critical flow. This should be located well before flow enters the bridge reach. Boundary conditions are always uncertain. The following is a brief summary for the initial estimation of the placement of the four cross sections.

- **First Cross Section**

Ideally, the first cross section should be located sufficiently downstream from the bridge so that the flow is not affected by the structure (i.e., the flow has fully expanded). This distance should generally be determined by field investigation during high flows and will vary depending on the degree of constriction, the shape of the constriction, the magnitude of the flow, and the velocity of the flow. In order to provide better guidance to determine the location of the fully expanded cross section, a study was performed by the Hydrologic Engineering Center and recommend Hunt and Brunner's equation [HEC-1995].

- **Second Cross Section and Third Cross Section**

The Second Cross Section and Third Cross Section used by the program to determine the energy losses through the bridge is located a short distance downstream and upstream of the structure respectively.

For this thesis, the cross sections are located at the toe of the roadway embankment on the downstream and upstream side of the bridge. The program will superimpose the bridge geometry onto those cross sections to develop a cross section inside the bridge at the downstream and upstream end

- **Fourth Cross Section.**

The fourth cross section is located upstream from the bridge where the flow lines are parallel and the cross section exhibits fully effective flow.

For this thesis, contraction ratio 1:1 and expansion ratio 4:1 was initially used for section 4 and 1 respectively. After the pressure/weir flow analysis, the location of this cross section was evaluated.

Since backwater, rise of water level caused by structure, is a key parameter in the hydraulic design of bridges and aggravates existing floods. Therefore, extending upstream cross-sections well beyond the bridge reach help to determine the effects of bridge backwater.

The cross-sections were aligned normal to the direction of flow and located with respect to each other as well. The numerical value was given for each section, HEC-RAS uses those numerical values to arrange the sections in the proper order. The length from each section to the next section immediately downstream was inserted in cross section data editor. These downstream reach lengths were entered separately for the left floodplain, right floodplain, and channel portion of the section. HEC-RAS requires that the top of bank stations be known for the main channel in each section. This is done in order to divide the section into three parts: right floodplain, channel, and left floodplain

3.2.4 Roughness coefficients and Loss coefficients

HEC-RAS also need a minimum of three roughness coefficients for each of the three parts of the cross-section (left bank, main channel and right bank). The value of roughness coefficient (in the form of Manning's n) depend on selected site and it also depends heavily on engineering experience. The value of roughness coefficients are selected from ERA drainage design manual 2013 table See manning values in appendix 3

HEC-RAS hydraulic model is developed based on basic hydraulic equations or scientific rather than empirical. But as discussed in above, the value of roughness depends heavily on engineering experience. It is also very important for Manning's n values to be calibrated whenever high water marks are available. But data were not available for Wedeba, Hamessa-1 and Raya River bridges for calibration and discharge data for Bishan Guracha and Baso River bridges were taken from upstream. Therefor sensitivity of manning coefficients were done by using minimum and maximum value of roughness coefficients to check the error. It show maximum error of 0.1m (See appendix 4).

Each cross-section must have loss coefficients for contraction and expansion. The contraction and expansion coefficients are used by the program to determine the transition energy losses between two adjacent cross sections. The user may provide any value desired ranging from 0.0 for no losses to 1.0 for maximum losses. From the data provided by the recent HEC study [HEC-1995], gradual transition contraction and expansion coefficients are 0.1 and 0.3, and typical bridge contraction and expansion coefficients are 0.3 and 0.5, respectively. For situations near bridges where abrupt changes are occurring, the coefficients may take larger values of 0.5 and 0.8 for contractions and expansions, respectively. For this study different values of contraction and expansion coefficients was examined.

3.2.5 Bridge Geometry Data

HEC-RAS provides a bridge editor to facilitate data entry (Figure below). The required data, to fill bridge editor were obtained from field survey and site inspection. New river station was inserted with respect to other cross-sections to locate the bridge. Once the user locates the bridge, HEC-RAS chooses the cross-sections that will serve as the BD (Bridge downstream) and BU (Bridge upstream) section and overlays all bridge data on them.

After bridge station was represented, data that describe the distance from the upstream side of the bridge deck to the cross section immediately upstream from the bridge, bridge deck width and a weir flow coefficient was inserted in Bridge/culvert data editor. Then input data that describe encroachment of the approach roadway on the floodplain was inserted. As with cross-sections, this is input in the form of stations and corresponding roadway elevations. The stations must be consistent with the stations of the cross-sections near the bridge. HEC-RAS then overlays the roadway data onto the cross-section data at sections BD and BU.

Finally data that describe location and thickness of all bridge piers and abutments, embankment slop, bridge deck and low chord elevations were inserted in Deck/Roadway data editor.

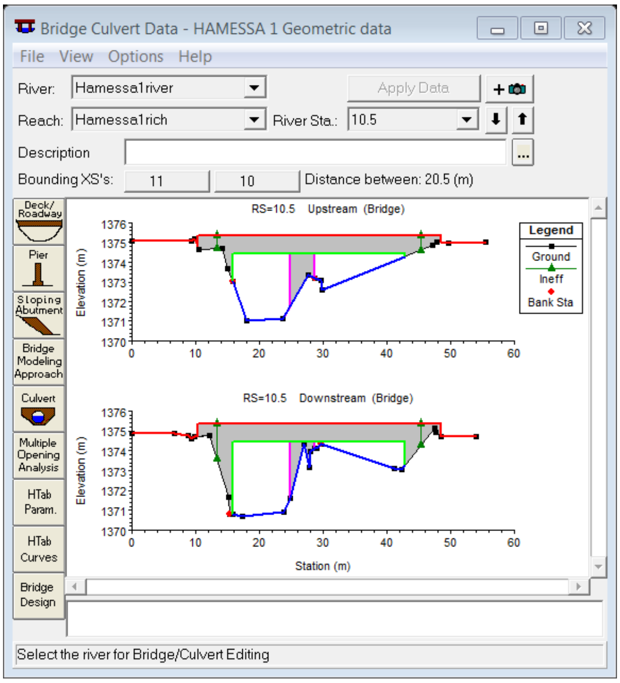

Figure 3-4: Bridge/culvert data editor (HEC-RAS software)

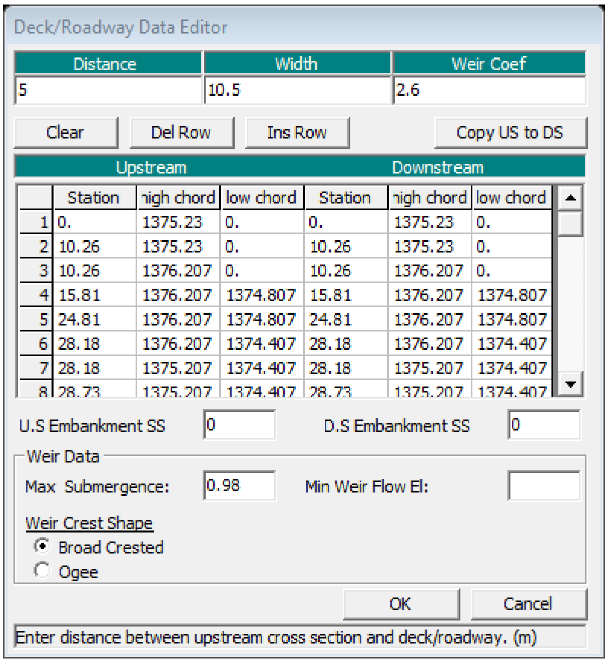

Figure 3-5: Deck/Roadway data editor (HEC-RAS software)

3.2.6 Flow Data

The main problem in analysis was the lack of flow data. Since many of the streams in Ethiopia are not gauged, the peak flood discharge that will be used for the drainage design cannot be estimated from recorded stream flow data. Peak flood discharge estimated from SCS method was used. This method is developed by the U. S. Soil Conservation Service for calculating rates of runoff and requires basic data's: catchment area, a runoff factor, time of concentration, and rainfall.

➢ **Catchment Area Delineation and Watershed Parameters**

Catchment areas that drain runoff to the road alignment were carefully delineated on topographic maps. Catchment delineation is shown in Appendix 1.2

Other catchment parameters such as average stream slope, length of longest watercourse and elevation difference were determined from topographic map scale 1:50,000 and 1:250,000. The following table presents some characteristics for large catchments.

Table 3-2 Summary of Catchment Characteristics for Large Catchments

Area No.	*Catchment Area (km^2)*	*Length of Main Stream Channel*	*Minimum Elevation (m)*	*Maximum Elevation (m)*	*Average Slope (%)*

A_3	*206.8*	*27.6*	*1690*	*2470*	*7.0*
A_{33}	*81.6*	*20.0*	*1850*	*2065*	*1.9*
A_{93-1}	*271.8*	*36.0*	*1350*	*2950*	*10.3*
A_{125}	*99.6*	*15.8*	*1250*	*1700*	*6.7*
A_{188}	*151.5*	*24.6*	*1150*	*2850*	*8.1*

- **Time of Concentration (Tc)**

Tc is defined as the time required for surface runoff water to flow hydraulically from the remotest point of the catchment to the point of exit. Tc comprises of summation of flow durations in sheet flow, shallow concentrated flow and open channels. The methods adopted to determine Tc are as follows:

- ✓ As per ERA DDM (Ethiopian Road Authority Drainage Design Manual) sheet flow condition is limited to a maximum of 100m and flow duration is computed using the simplified Manning kinematic solution.
- ✓ For shallow concentrated flow, the velocity method (Upland method) is used.
- ✓ For flow in open channels, the Manning' equation is used.

In using the above procedures for determination of Tc, the following difficulties may be encountered:

- ✓ In sheet flow computation, although a maximum limit of 100m is stated, an accurate demarcation of flow length is somehow difficult and may not be accurate.
- ✓ For shallow concentrated flow, the stretch (length) of the flow is not readily determined from topographical maps or as aerial photos.
- ✓ Use of the Manning's equation for open channel flows is dependent on availability of channel geometric section properties, which once again is difficult to determine from topographic maps and aerial photos. This may require conducting channel cross section surveys *at various locations* along the stream which is practically difficult especially on large catchment areas.

Hence, due to the above conditions, Kiprich's equation employed, noting the caution stated in the ERADDM. In order to minimize in estimating too short Tc, in using this equation, the channel is subdivided into a number of stretches with similar slopes and Tc for each stretch is calculated and summed up to obtain the final Tc. A maximum number of 3 stretches have been used for a given water course and the detailed Tc computation for each respective stretch is shown in Appendix 1.4

Kirpich's equation is given as:

$$T_c = \frac{0.06628L^{0.77}}{S^{0.385}}$$

Where: T_c *= Time of concentration (hr)*

L = Maximum length of travel (Km)

S = the mean channel slope (m/m)

- **Curve Number (CN)**

CN is generally estimated from a classification in one of four hydrologic soil groups (A, B, C and D) depending on infiltration rate and permeability capability together with a hydraulic condition (poor, fair and good) and land use (ground cover). For details of types and descriptions, reference is made to Table 5-8 to 5-13 of ERA DDM.

Hydrologic soil grouping is a classification by the Soil Conservation Service (SCS), which is based on permeability and infiltration rate. Hydrologic soil grouping for each catchment is identified from examination of available soil maps of scale 1:1,000,000 and physical assessment done on site. Accordingly, the hydrological soil group is found to be mostly 'Type B', while the rest of the areas are 'Type D'. Summary of hydrological soil grouping of the project is tabulated in Appendix 1.3

- **Return Periods**

The frequency of the flood for the design of drainage structures depends on the risk likely to be encountered during the anticipated service life of the road. Return period with which a given flood can be expected to occur is the reciprocal of the probability or chance that the flood will be equaled or exceeded in a given year. The Bridges have been designed for recurrence interval as per Drainage Design Manual of ERA recommendation.

Table 3-3 Return Period Based on the Size of Catchments and Type of Structures

Drainage Facility Type	*Description*	*Return Period*
Bridge	*Short span bridge, 6m<span<15m* *Drainage Area<50Km²*	*50*
	Medium span bridge, 15m<span<50m *Drainage Area<200Km²*	*50*
	Long span bridge, span>50m	*100*

- **Rainfall Depth (P)**

24hr rainfall data was available for four locations, i.e. at Alaba, Wolaita Sodo, Mirab Abaya and Arbaminch. These data set provided annual series of 24hr rainfall depth that is used in extreme value frequency analysis and for which the Gumble and Log-Pearson III distributions has been utilized. For the project, the Log-Pearson III distribution has been adopted since the data sets have been found to have better fitting.

Basically, the steps followed during Log-Pearson III Distribition are as follows.

- ✓ From peak discharges (X) for each water year are listed.
- ✓ Logarithmic transformation for each value is carried out, Y=Log(X).
- ✓ Mean (Ym) and standard deviation (Sy) of the logarithmic values are carried out.
- ✓ Standardized skew (Cs) of the logarithms is calculated using the following expression.

  $Cs = n.\sum(Y - Ym)^3/(n-1).(n-2).Ys^3$

- ✓ Frequency Factor (Kt) are read based on standardized skew (Cs) and return period.
- ✓ Logarithmic value Yt is determined using the formula below for each return period.

  $Yt = Ym + (Sy * Kt)$

- ✓ Flood Flow (Xt) is determined by transforming values of step 4 using

  $Xt=10^{Yt} = P$

Details of the analysis are shown in Appendix 1.1

➢ Peak Discharge Computation

For the return periods determined, the peak design discharge and review peak discharges were computed by using United States Soil Conservation Service (SCS) Method.

A summary of Hydrological analysis steps is shown on the following flow chart. Details of the analysis are shown in Appendix 1.6.

A
- Determine the watershed area in hectares (km2)
- from topographic maps; DEM data's and field surveys

Tc
- Determine the time of concentration Tc (hr)
- with consideration for future characteristics of the watershed

CN
- Determine Curve number
- based on land land cover and soil type

P
- Determine retern pered and 24hr rainfall depth with P(mm)
- based on land land cover and soil type

S_r
- Potential Maximum retention, Sr (mm)
- $S = \frac{25400}{CN} - 25$

I_a/P
- Determine prametre I_a/P
- where intil abstrsction, I_a (mm) = $0.2*S_r$

Q
- Determine Accumulated Direct Runoff, Q (mm)
- from graph (ERA DDM, 2002). Q f(P,CN)

q_u
- Determine Unit Peak Discharge, qu (m3/s)
- from graph (ERA DDM, 2002). q_u f(Tc,I_a/P)

q_p
- Calculate the peak discharge for the watershed for the desired frequency.
- $q_p = q_u * A * Q$

From HEC- RAS main window Steady Flow Data were selected to enter the steady flow data. This activated the Steady Flow Data Editor as shown in Figure 3.6. Single profiles were selected for each river to be computed. The flow data were entered for the upstream station and flows are continuous throughout the reach so no flow change locations were used.

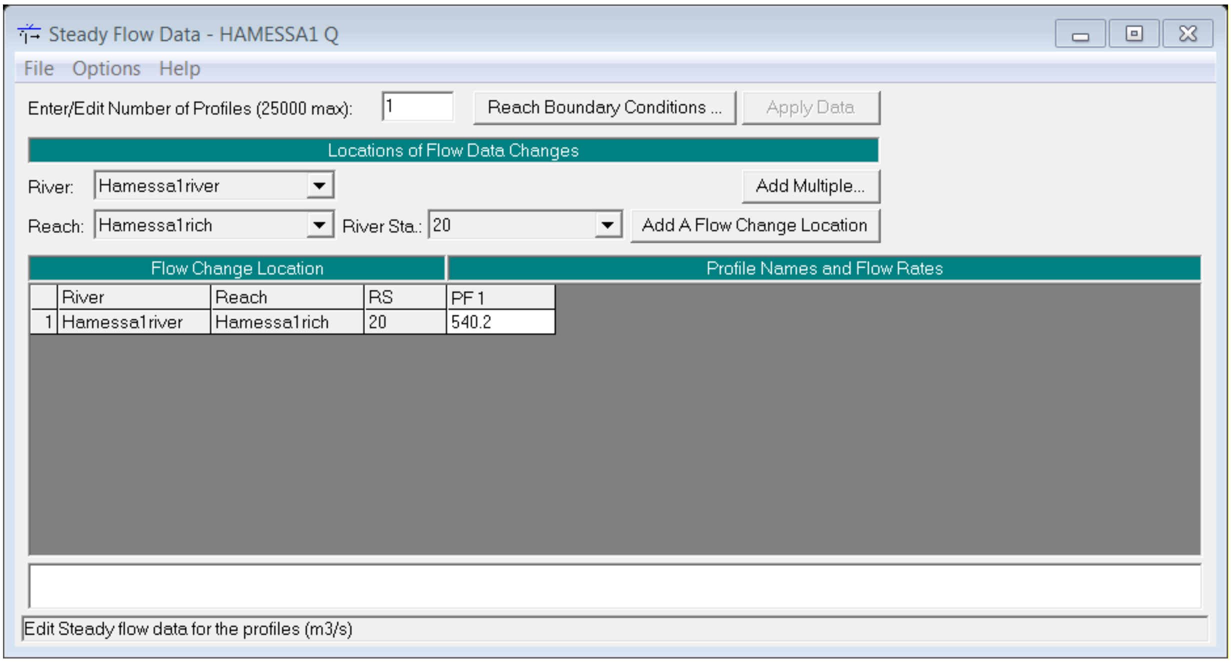

Figure 3-6: Steady Flow data editor (HEC-RAS software)

Boundary condition is required for each flow value. To enter the boundary conditions, the Reach Boundary Conditions button is selected and then, 1 of the 4 boundary conditions was selected. Multiple runs were performed to observe the effect of changing the boundary conditions on the output of the main area of interest.

3.2.7 Ineffective Flow Areas

Ineffective flow areas also entered as station and elevation be in cross-sections within the bridge reach. Ineffective flow areas were used to define an area of the cross section in which the water will accumulate but is not being actively conveyed. At a bridge ineffective flow areas normally occur, just upstream and downstream of the road embankment, away from the bridge opening, within contraction and expansion reach. See Figure 2.1 in the previous chapter for graphical illustration of contraction and expansion reach.

To determine the initial elevation of the ineffective flow areas for the upstream cross section, a value slightly lower than the lowest high cord elevation was used. This ineffective flow elevation was chosen so that when the water surface becomes greater than this ineffective elevation, the flow would most likely be weir flow and would be considered as effective flow. At the downstream cross section, the elevation of the ineffective flow area was set to be slightly lower than the low cord elevation. This elevation was chosen so that when weir flow occurs over the bridge, the water level downstream may be lower than the high cord, but yet it will contribute to the active flow area.

3.2.8 Bridge Modeling Approach

As discussed in pervious chapter HEC-RAS allow the modeler to analyze the bridge flows by using different methods with the same geometry. The different methods are: low flow, high flow, and combination flow. Low flow occurs when the water only flows through the bridge opening and is considered as open channel flow (i.e., the water surface does not exceed the highest point of the low cord on the upstream side of the bridge). High flow occurs when the water surface encounters the highest point of the low cord on the upstream side of the bridge. Finally, combination flow occurs when both low flow and pressure flow occur simultaneously with flow over the bridge. For the combination flow, the program will use the methods selected for both of the flows.

As shown figure 3.7 Low Flow Method contain four methods and high flow method contain two methods.

For this paper we have two high flow and three low flow scenarios. All methods are applied and comparison is made.

4 RESULTS AND DISCUSSION

After all data was input as discussed previously for five bridges and several important parameters were varied in order to determine water surface profile for each method or technique discussed below. Then comparisons were made.

4.1 Transition Lengths

The first parameters to be analyzed were the contraction and expansion lengths. RAS, HEC-2, and WSPRO all give separate recommendations for these values as previously discussed. Each bridge flood event was modeled using contraction and expansion lengths determined by each recommendation. Contraction and expansion coefficients were 0.1 and 0.3 respectively outside of the bridge reach while 0.3 and 0.5 were used within the bridge transition zones. A base condition with no transition reaches was also modeled. The no transition condition was developed without ineffective flow areas and with contraction and expansion coefficients of 0.1 and 0.3 at all cross-sections.

For each bridge water surface elevation is analyzed by HEC-RAS using energy method for low flow event and pressure and weir method for high flow events. Even though we don't have observed water surface elevations to decide which method is applicable to our site, the analysis tell us the effect of reach length in water surface profile. Table 4.1 show the maximum difference of water surface profile for the various methods of contraction and expansion lengths.

Table 4-1 Range of Errors for the various methods

	Min and max error between with & without transition	*Min and max error between HEC2 & RAS*
Wedeba	*0.01 - 0.16*	*0.016 - 0.12*
Bishan Guracha	*0.04 - 0.11*	*0.06 - 0.14*
Baso	*0.019 - 0.08*	*0*
Hamessa 1	*0.01 - 0.08*	*0.05 - 0.07*
Raya	*0.01 - 0.47*	*0.03*

As Table 4.1 indicates, water surface elevation computations show maximum error of 0.47m, in case of Raya River Bridge, if we exclusion of bridge transition reaches and maximum error of 0.14m, for case of Bishan Guracha, between HEC-2 and RAS recommendation.

Water surface elevation computations performed exclusion of bridge transition reaches for downstream expansion and upstream contraction of flow result in calculated water surface elevations which are much lower than others. This method does not account for energy losses due to expansion and contraction, and calculated elevations are too low. Designers should use both expansion and contraction reaches.

Water surface profile calculated using HEC-2 recommends, exit section be placed four times the obstructed length downstream, and RAS, Regression equations developed by Hunt and Brunner (1995), show average 0.13m difference. There for it is better to use the highest value (HEC-2) recommendation bay pass to unexpected increasing of aggradation and unexpected blockage.

4.2 Influence of Boundary Conditions

All hydraulic computations require a beginning boundary value. This value is then used to begin progression of the standard step method along the river reach. Calculations begin at the downstream most cross-section and proceed upstream for subcritical flow and begin at the upstream most cross-section and proceed downstream for supercritical flow.

Three flood events were analyzed to determine the effects of boundary values upon computational accuracy. Each event was analyzed using normal depth, critical depth, and observed water surface elevation for boundary values. The iterative nature of the standard step method causes profiles computed with each boundary value to converge as computations move upstream. This is illustrated in Figure 4.1, 4.1 and 4.3.

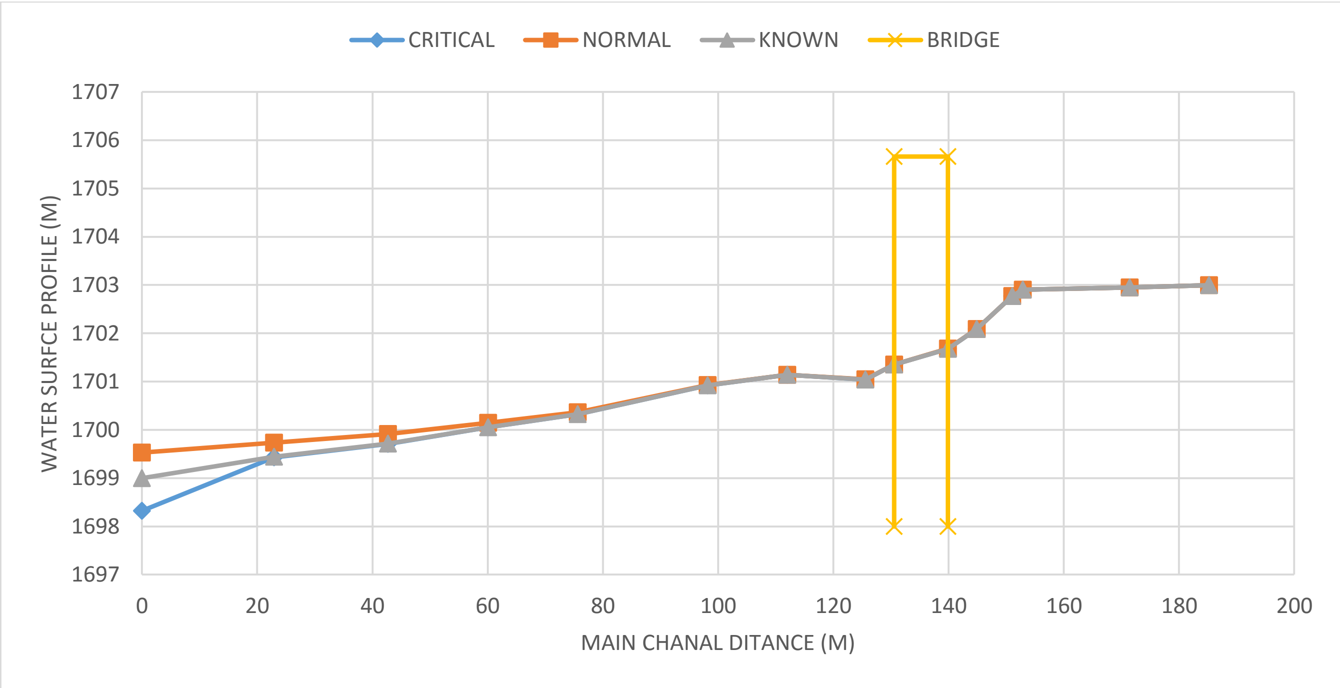

Figure 4-1:Bishan guracha Water surface profiles for various boundary conditions.

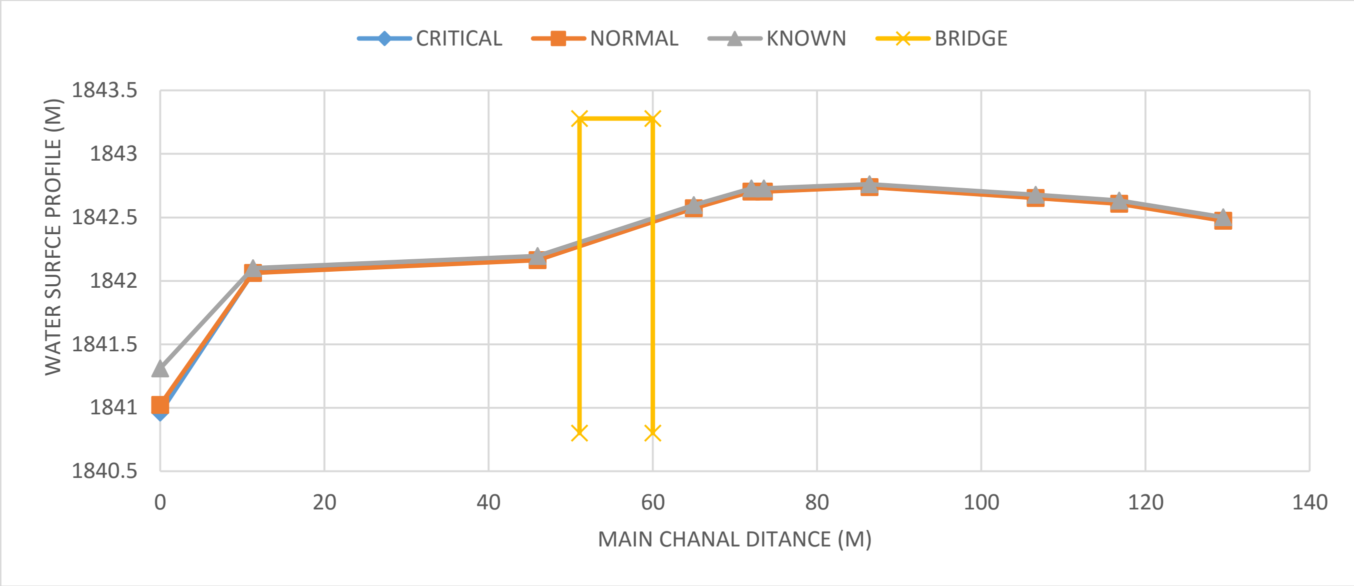

Figure 4-2: Wedeba Water surface profiles for various boundary conditions.

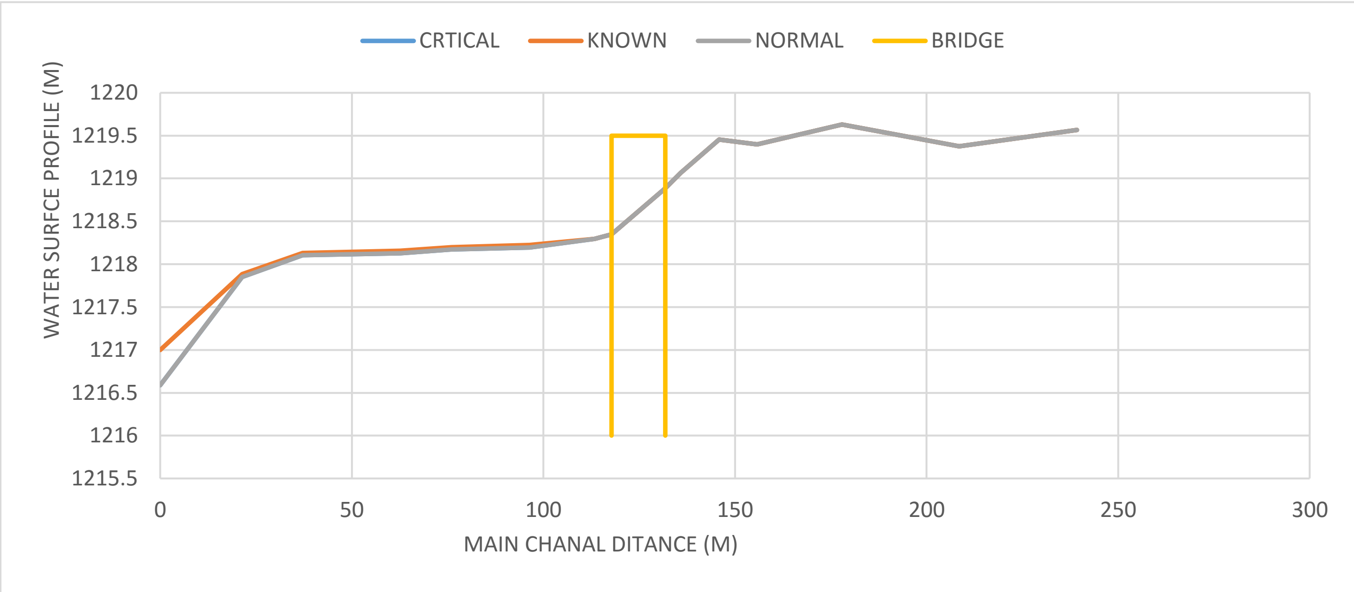

Figure 4-3: Raya Water surface profiles for various boundary conditions.

The result shows poorly chosen boundary values will affect computed water elevations. But due to the iterative nature of the standard step method causes profiles computed with each boundary value to converge as computations move upstream. Movement of computations along the profile slowly eliminates errors as each new cross-section computation is closer to the actual. At some point profiles computed with various boundary values converge and computations are no longer dependent upon boundary conditions. So to eliminate errors quickly designer should use smaller spacing, especially when there is lack data for validation and calibration of models.

4.3 Bridge Analysis Methods

HEC-RAS allow the modeler to analyze the bridge flows by using different methods. Four methods for computing water surface elevations at bridges during low flow (energy, momentum, Yarn ell, and WSPRO) and two methods during high flow method (energy method and pressure/weir method).

For each bridge water surface elevation is analyzed using different methods and comparison is made. Table 4.2 show summary of water surface profile for the various methods

Table 4-2 Upstream water surface profile of each bridge for the various methods

BRIDGES	*W.S. U/S.(m)*	*BR Sel. Method*	*Energy WS(m)*	*Momen. WS(m)*	*WSPRO WS(m)*	*Yarnell WS(m)*	*Energy WS(m)*	*Prs/Wr WS (m)*
Bishan Guracha	*1702.44*	*Momentum*	*1702.09*	*1702.44*	*1702.04*	*1701.04*	-	-
Wedeba	*1842.57*	*Energy only*	*1842.57*	-	*1842.57*	-	-	-
Raya	*1219.07*	*Energy only*	*1219.07*	-	-	-	-	-
Baso	*1189.57*	*Press/Weir*	-	-	-	-	*1189.82*	*1189.57*
Hamessa 1	*1380.09*	*Energy only*	-	-	-	-	*1380.09*	*1380.06*

The differences in computed water surface elevations appears to be primarily due to pier losses. The energy and WSPRO methods compute water surface elevations by an energy based approach. In this method piers simply reduce available area for flow and add wetted perimeter. The Yarnell method does account for piers to some extent, but ignores area of the bridge opening, and the bridge itself. The momentum equation computes pier losses as a function of flow and area. Since velocity is also a function of flow and area, then pier loss is a function of velocity.

HEC-RAS documentation (HEC, 1997) makes this recommendation: "In cases where pier losses and friction losses are both predominant, the momentum method should be the most applicable, but any of the methods can be used."

Poorly chosen Bridge modeling method at bridges can result in extremely large errors. About 1.4m in the case of Bishan Guracha. Care must be taken when choosing modeling method. In the case of low flow, the momentum method should be applied if both pier losses and friction losses are predominant or velocity is high through the bridge.

4.4 Adequacy of the Bridges opening and Back Water Analysis

Initially using an existing data (geometric and flow) water surface profiles were produced for all bridges by mean of computer modeling technique using HEC-RAS software. Modelling approach is chosen based on highest energy.

Backwater water computation is performed and the propagation of backwater is checked if it is within appropriate limit, i.e. does not cause damage to upstream property. Then, after checking this effect, the opening was declared either as adequate or inadequate with the primary aim of accommodating the design flood in a safe and economic manner.

4.4.1 Wedeba River Bridge

After all data entered in to HEC-RAS steady flow analysis is run to compute water surface profile. The result show Wedeba River Bridge has 1.05 m Vertical clearance and 0.39 m Back water rise. Table 4-3 shows summary of the results obtained in this regard from the HEC-RAS and figure 4.4 show vertical clearance in the U/S side of Wedeba River Bridge. Since vertical clearance is 1.05m and Back water rise is within acceptable range Wedeba River Bridge is hydraulically safe.

Table 4-3 Wedeba Bridge summary of the results obtained from HEC-RAS.

Manning coefficient	*0.045 for both chanal and banks*
Modelling approach	*Energy method*
Vertical clearance	*1.05m*
Back water rise	*0.39m*

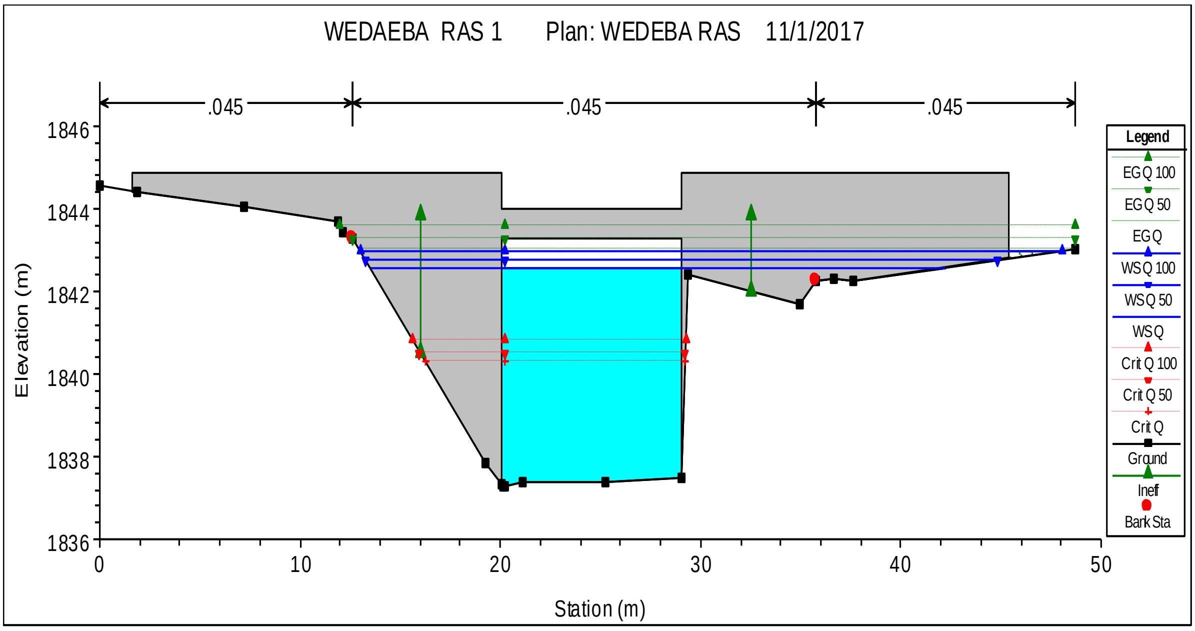

Figure 4-4: Upstream water surface profile of Wedeba bridge

4.4.2 Bishan Guracha River Bridge

All necessary data were inputted in to HEC-RAS hydraulic model to simulate steady flow. Table 4-4 shows summary of the results obtained in this regard from the HEC-RAS and figure 4.5 show vertical clearance in the U/S side of Bishan Guracha River Bridge. As show in table 4.4 Bishan Guracha River Bridge has 2.75 m Vertical clearance and 1.35 m Back water rise. Bishan Guracha River Bridge has enough opening, which is 2.75 m Vertical clearance (above H.W.L), to accommodate design discharge but higher value of backwater height about 1.34m. Since Bishan Guracha River is located in rural area with little or no development, no major damage is expected.

Table 4-4 Bishan Guracha Bridge summary of the results obtained from HEC-RAS.

Manning coefficient	*0.085 for both channel and banks*
Modelling approach	*Momentum*
Vertical clearance	*2.75 m*
Back water rise	*1.1 m*

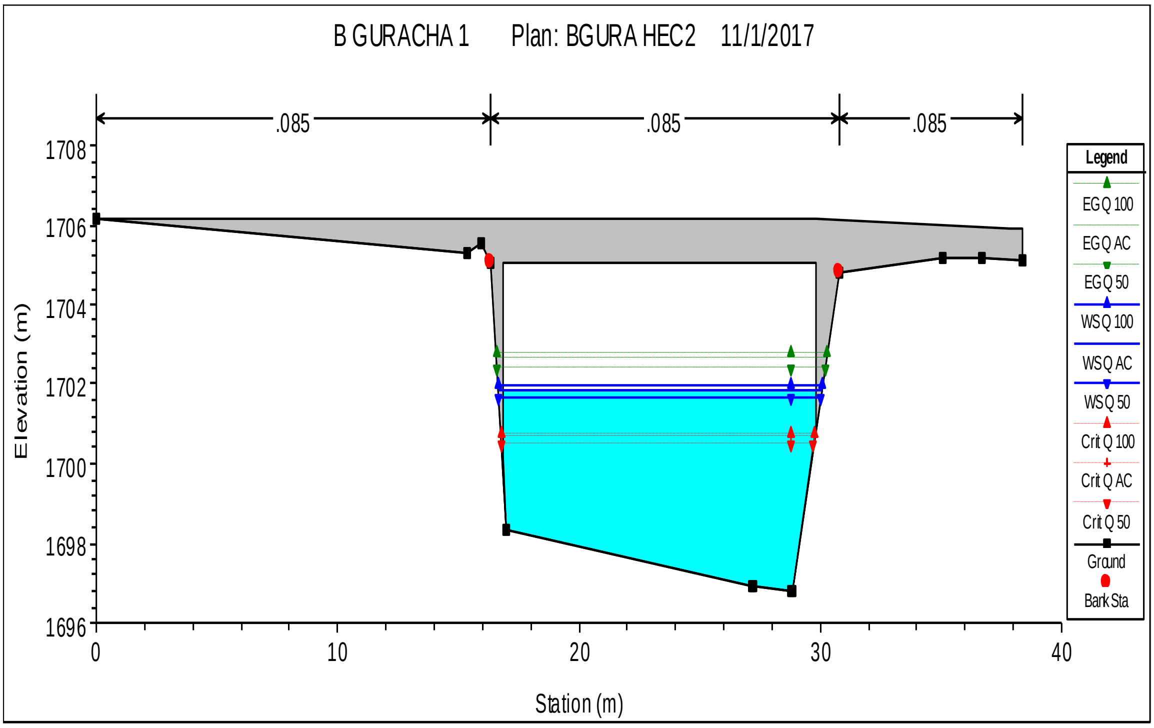

Figure 4-5: Upstream water surface profile of Bishan Guracha bridge

4.4.3 Baso River Bridge

The result from HEC-RAS hydraulic model for Baso River Bridge summarized in Table 4-6 and figure 4.7. As shown in the figure Baso River Bridge has no enough height to accommodate design flood. The flood over floats one of the super structure of the bridge resulting collapse.

Table 4-5 Baso River Bridge summary of the results obtained from HEC-RAS.

Manning coefficient	*0.03 for both channel and banks*
Modelling approach	*Press/Weir*
Vertical clearance	*Over flow*
Back water rise	-

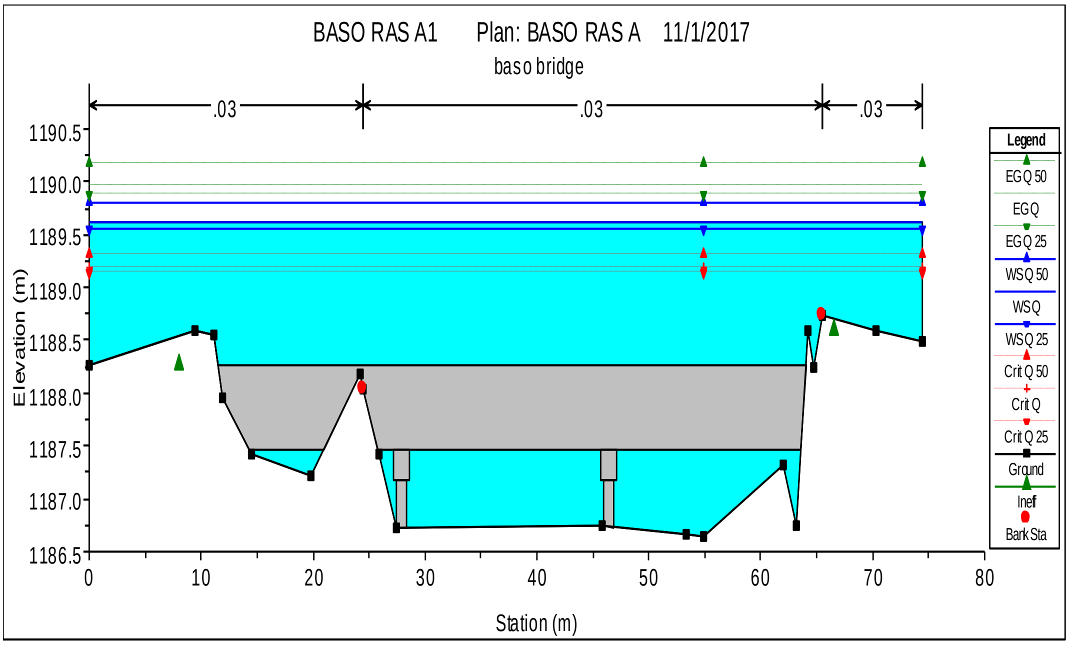

Figure 4-6: Upstream water surface profile of Baso Bridge

The survey data's obtained from ERA BMS show that Baso river bridge opening area reduced by 1.7 m depth in each year due to aggradation.

For Baso rivers bridges serious and urgent countermeasure need to be exercised. The opening currently very small (less than 1m) and have persistent aggradation problem. This condition is aggravated by human induced activities such as deforestation on upstream of watershed. In addition to this at the bridge locations there is no sufficient slope (energy) to transport the incoming excess sediment (see Figure 4.8)

Figure 4-7 Baso under view

Therefore, only dredging activities didn’t last one major flooding event. In order to give long term solution detail investigation of watershed management should done to prevent this enormous amount sedimentation.

4.4.1 Raya River Bridge

All necessary data were inputted in to HEC-RAS hydraulic model to simulate steady flow. Table 4-5 shows summary of the results obtained in this regard from the HEC-RAS Hydraulic model and figure 4.6 show vertical clearance in the U/S side of Raya River Bridge. thr result show that Raya River Bridge has 0.2m Vertical clearance, it need attention, and 0.55 m Back water rise.

Monitoring and hydraulics performance of bridges were done at different time by ERA Bridge Management System (BMS). The amount of sediment deposited around the bridge location were analyzed by using the collected survey data’s of the river bed level changes. Based on the obtained cross section data’s within the last three years Ray river bridge opening reduced by 0.6 m depth in each year. The data has been clearly demonstrates that the cross section area of the bridge was dynamically changed due to the enormous amount of sediment deposition around the bridge location.

For the Raya River Bridge there is a need to perform periodic dredging work within two years interval. Otherwise if this problem happens and is not treated in time, it may get worse, aggravate and will be further exacerbated. So, timely maintenance and intervention measures are very necessary.

Table 4-6 Raya River Bridge summary of the results obtained from HEC-RAS.

Manning coefficient	*0.03 for both channel and banks*
Modelling approach	*Energy method*
Vertical clearance	*0.2 m*
Back water rise	*0.55 m*

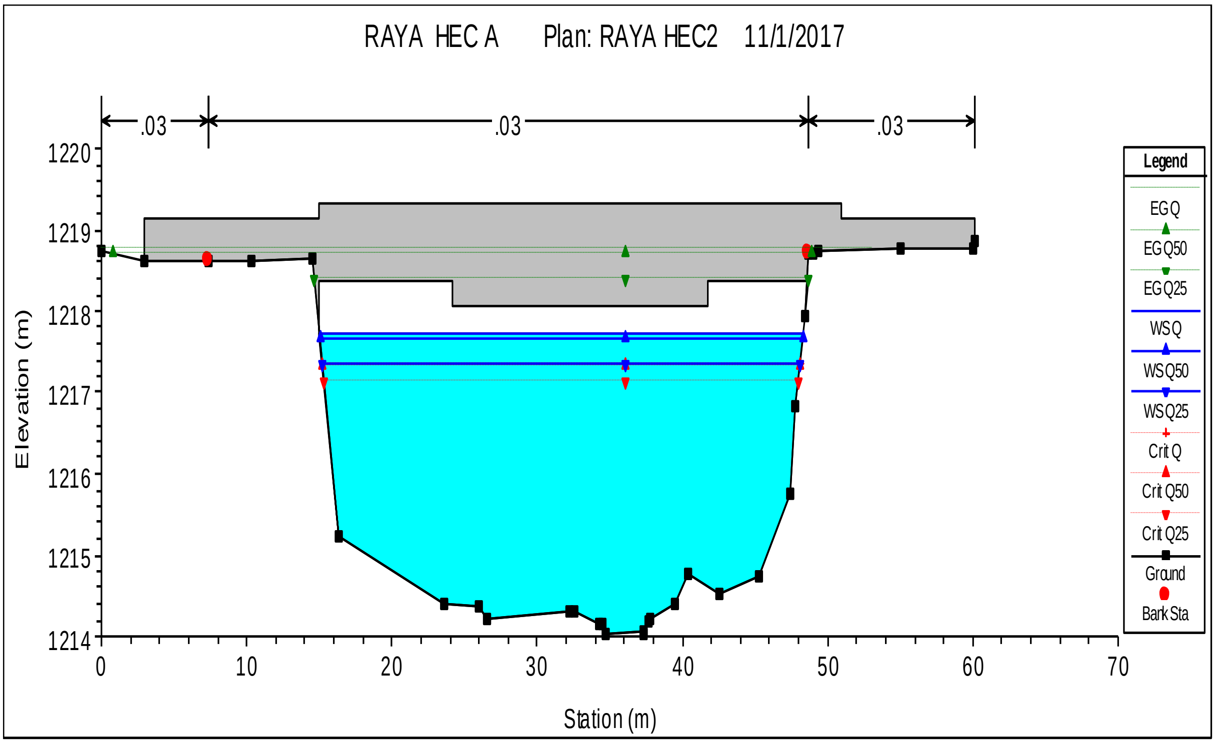

Figure 4-8: Upstream water surface profile of Raya bridge

4.4.2 Hamessa1 River Bridge

Hamessa-1 River Bridge also has no enough height to accommodate design flood. The flood over floats one of the super structure of the bridge resulting collapse. Table 4-7 shows summary of the results obtained in this regard from the HEC-RAS and figure 4.9 show vertical clearance in the U/S side of Hamessa-1 River Bridge.

Table 4-7 Hamessa-1 River Bridge summary of the results obtained from HEC-RAS.

Manning coefficient	*0.03 for banks and 0.035 for channel*
Modelling approach	*Energy only*
Vertical clearance	*Over flow*
Back water rise	-

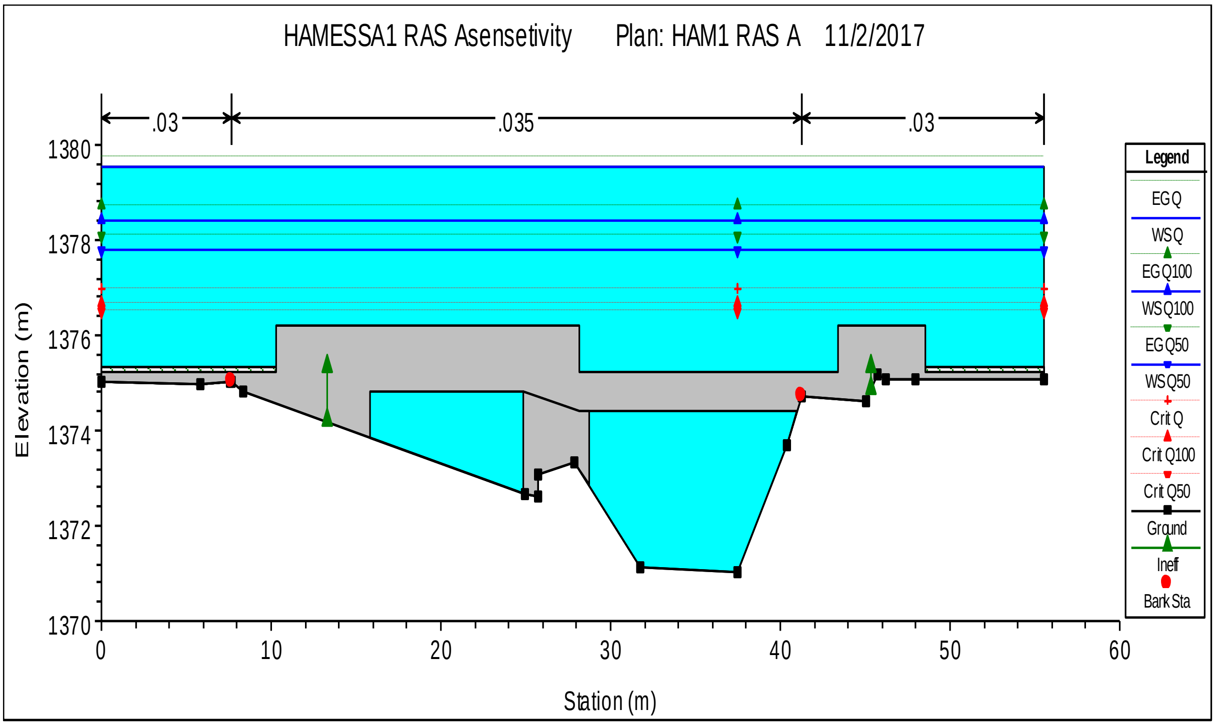

Figure 4-9: Upstream water surface profile of Hamessa1 Bridge

even though enough The flood over floats on super structure of the bridge, The case with Hamessa-1 is rather different in that its span and clearance is completely blocked on one side by silty clay material that is transported from the nearby agricultural land (see Figure 4.10 & Fig 4.11) If this obstacle is removed and further channelization is carried out both upstream and downstream of the river bridge it will bring about enough opening size. But this problem needs to be attended routinely. And also the river bank on the Arbaminch side is eroded which requires protection work.

Figure 4-10 Hamessa-1 inlet view

Figure 4-11 Hamessa-1 outlet view

5 SUMMARY, CONCLUSION AND RECOMMENDATION

5.1 Summary

One major cause for bridge failure is hydraulic problem. Since maintaining or reconstructing bridge require great amount of money, its negative impact on the economy of the country is high. These indicate a need for rigorous design procedures. HEC-RAS is the most recent software program developed to aid in design of hydraulic structures.

In this thesis five river bridges are analyzed using HEC-RAS software program to evaluate practice of bridge hydraulic design in our country. An effort was also made to evaluate the effect of using different methods and recommendations. The primary issues examined by this thesis were:

- ✓ Adequacy of the Bridges located in five river (Webeba, Bishan Guracha, Baso, Raya and Hammesa 1 river) were analyzed to determine whether they are safe or not.
- ✓ Effect of transition length recommended by HEC-2, HEC-RAS and a situation with no transitions.
- ✓ Influence of Boundary Conditions. Any reach analyzed with HEC-RAS requires a user-specified condition at the reach boundary. HEC-RAS provides three methods: critical depth, normal depth, and observed water surface elevation. The user must judge which of these is best for the situation being analyzed. Some guidance concerning the effects of boundary values upon the modeling computations was developed.
- ✓ Effect of using different Bridge Analysis Methods

5.2 Conclusion

The following conclusions were drawn from the results of the investigation:

Exclusion of bridge transition reaches for downstream expansion and upstream contraction of flow result in calculated water surface elevations which are much lower than others. This method does not account for energy losses due to expansion and contraction, and calculated elevations are too low. Designers should use both expansion and contraction reaches.

Water surface profile calculation using HEC-2 method and HEC-RAS regression equations developed by Hunt and Brunner (1995) recommends different things. HEC-2 method recommends that the exit section to be placed at distance of four times the obstructed length to the downstream while the HEC-RAS regression equation recommends other equation. HEC-2 recommendation averagely 0.13m higher than that of HEC-RAS regression equation. Therefore it is better to use the highest value recommended by HEC-2 as there may be unexpected increase in an aggradation.

On the other hand poor selection of the boundary value will also have an effect on water surface elevation calculation. However, iterative nature of the standard step method used for profile calculation causes profiles computed with each boundary to converge as computations move upstream. Thus this iterative nature of the method causes the errors to be eliminated at each new cross-section and the computation result became so closer to the actual. So it can be concluded that in order to eliminate errors as quickly as possible, the designer should use smaller spacing between the cross-section, especially when there is lack data for validation and calibration of models.

Furthermore, poorly chosen Bridge modeling method at bridges can result in extremely large errors. As it was shown above under section 4.3, about 1.4m error was happen for the case of Bishan Guracha. So, care must be taken while choosing the modeling method. In the case of flow, the momentum method should be applied if both pier losses and friction losses are predominant or velocity is high through the bridge.

Bishan Guracha and Wedeba River bridges area hydraulically safe but Raya, Baso and Hamessa1 River Bridges have not sufficient opening to accommodate design discharge. With this decreasing of bridge opening area, the flow property was changed to pressurized flow. As result the flood over float one of the super structure of the bridge resulting in bridge collapse. For the case of Baso River Bridge the main cause that resulted to the change in the property of the flow was the long term rise or aggradation of the river bed level due to sediment deposition. This makes the bridge opening very small (i.e. less than 1m currently) and also there was no sufficient slope for flow.

For Raya River Bridge the main cause are aggradation and the presence of hydraulic structure at the upstream of the bridge location which aggravates the rate of sediment deposition at the bridge location. Since ERA BMS recorded data shows that the opening of Raya River Bridge is reduced by 0.6 m depth in each year it need periodic maintenance.

Whereas the case with Hamessa1 River Bridge is rather different in that its span and clearance is completely blocked on one side by silty clay material that is transported from the nearby agricultural land. Removing blockage and further channelization will bring about enough opening size.

5.3 Recommendation

The following recommendations were drawn from this study

- ✓ As most of recorded bridge sites data analysis shows sedimentation is the main cause of the bridge failure. Thus it will indicate us intensive water shade management is required before and especially after bridge construction.
- ✓ For this thesis the method used for water surface elevation calculation was the one that resulted in the higher value of elevation, i.e. HEC-2 method, however it was strongly recommended to select different representative bridge sites and develop new manual or modify the old ERA manual with respect to the best model that specific area can fit with.
- ✓ Verifying bridge hydraulic design by model must be incorporated as criteria in ERA design manual to minimize hydraulic bridge failure.
- ✓ Further work is needed as an additional for different types of bridge at different sites as all the bridge sites discussed in this thesis are only located along the road from Alaba – Sodo – Arbaminch district.
- ✓ Furthermore sediment concentration flow data collection is also required to estimate the exact amount of sediment deposition around the bridge.
- ✓ In our country lots of bridge failures were experienced and researchers have to do a lot of works in this topic.

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APPENDIX

Appendix 1 - Hydrological analysis

Appendix 1.1 - Summary of Extracted Maximum Daily Rainfall Data and Analysis Based on Log Pearson Type III Method

Annual Maximum 24 hr Rainfall (mm)				
Year	Alaba Kulito	Wolaita Sodo	Mirab Abaya	Arbaminch
1980	43.8			
1981	47.3			
1982	36.7			
1983	66.4			
1984	83			
1985	74.5			
1986	60.3			
1987	68	58.3		39
1988	62	42.6		46.9
1989	64	71		72.1
1990	68	73.3	44.5	39.3
1991	56.4	42.8	78.5	54
1992	74.7	43.8	42.8	71.2
1993	94.5	78.5	89	70
1994	37.9	52.3	65.2	38
1995	52.4	55	96.2	40.7
1996	50	63	49	66.8
1997	75.4	47.3	75	68.2
1998	48.2	63.2	76	46.6
1999	59	40.6	59.5	75.3
2000	45.4	79.2	56	54.3
2001	57	83.8	56.6	48
2002	48.4	42.5	54.5	40.6
2003	55.4	54.2	70.9	46.8
2004	55	63.8	56.2	38.8
no. of Years	25.00	18.00	15.00	18.00
Ÿn	59.35	58.62	64.66	53.14
αn	14.04	14.14	15.87	13.64

	Alaba Kulito		Wolaita Sodo		Mirab Abaya		Arbaminch	
Year	Y=Log X	$(Y\text{-}Ym)^3$	Y=Log X	$(Y\text{-}Ym)^3$	Y=Log X	$(Y\text{-}Ym)^3$	Y=Log X	$(Y\text{-}Ym)^3$
1980	1.64	0.00						
1981	1.67	0.00						
1982	1.56	-0.01						
1983	1.82	0.00						
1984	1.92	0.00						
1985	1.87	0.00						
1986	1.78	0.00						
1987	1.83	0.00	1.77	0.00			1.59	0.00
1988	1.79	0.00	1.63	0.00			1.67	0.00
1989	1.81	0.00	1.85	0.00			1.86	0.00
1990	1.83	0.00	1.87	0.00	1.65	0.00	1.59	0.00
1991	1.75	0.00	1.63	0.00	1.89	0.00	1.73	0.00
1992	1.87	0.00	1.64	0.00	1.63	0.00	1.85	0.00
1993	1.98	0.01	1.89	0.00	1.95	0.00	1.85	0.00
1994	1.58	-0.01	1.72	0.00	1.81	0.00	1.58	0.00
1995	1.72	0.00	1.74	0.00	1.98	0.01	1.61	0.00
1996	1.70	0.00	1.80	0.00	1.69	0.00	1.82	0.00
1997	1.88	0.00	1.67	0.00	1.88	0.00	1.83	0.00
1998	1.68	0.00	1.80	0.00	1.88	0.00	1.67	0.00
1999	1.77	0.00	1.61	0.00	1.77	0.00	1.88	0.00
2000	1.66	0.00	1.90	0.00	1.75	0.00	1.73	0.00
2001	1.76	0.00	1.92	0.00	1.75	0.00	1.68	0.00
2002	1.68	0.00	1.63	0.00	1.74	0.00	1.61	0.00
2003	1.74	0.00	1.73	0.00	1.85	0.00	1.67	0.00
2004	1.74	0.00	1.80	0.00	1.75	0.00	1.59	0.00
Mean (Ym)	1.76	0.00	1.76	0.00	1.80	0.00	1.71	0.01
Standard Deviation (Sy)	0.10		0.10		0.11		0.11	

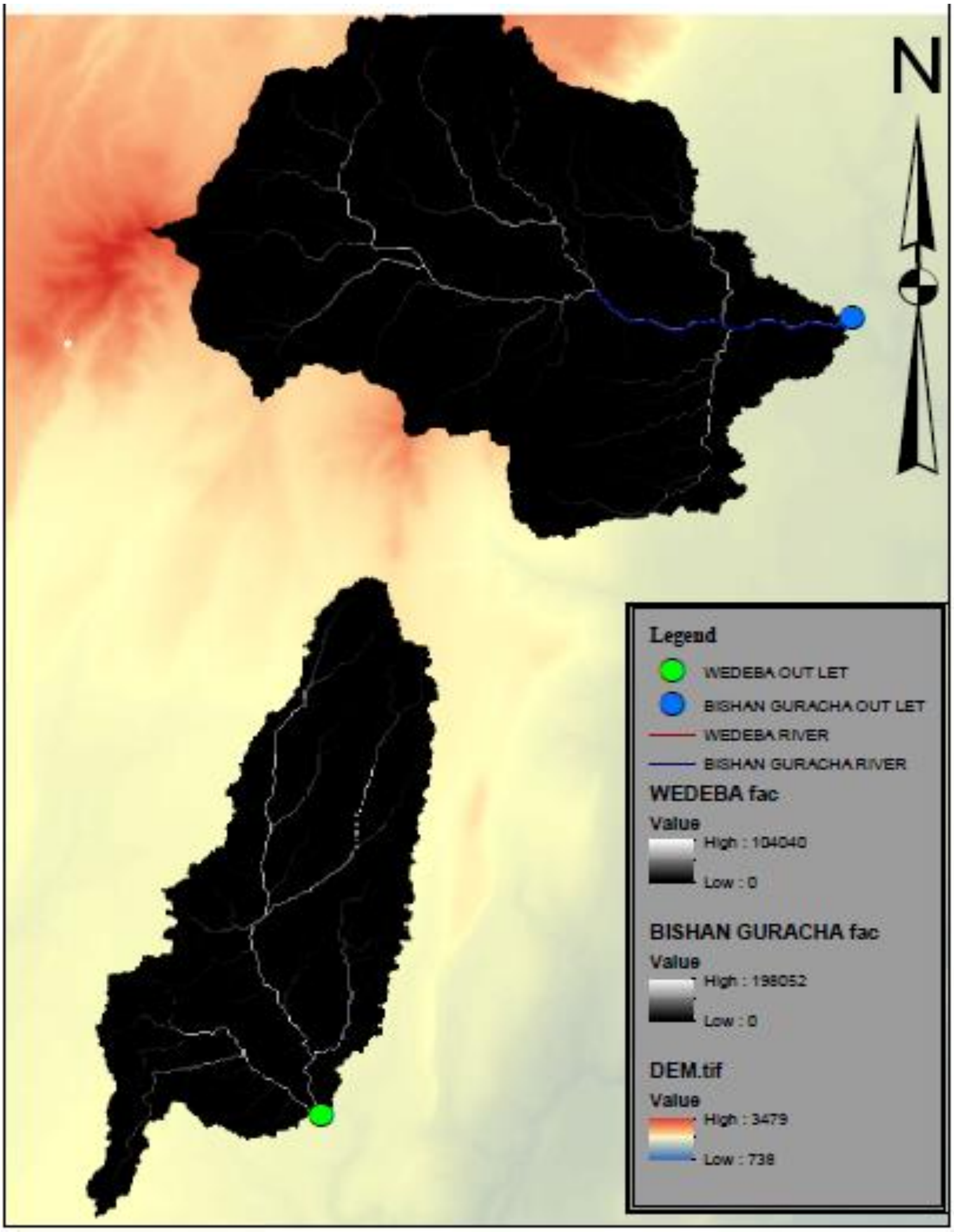
$Y_T=Y_m+K_TS_y$

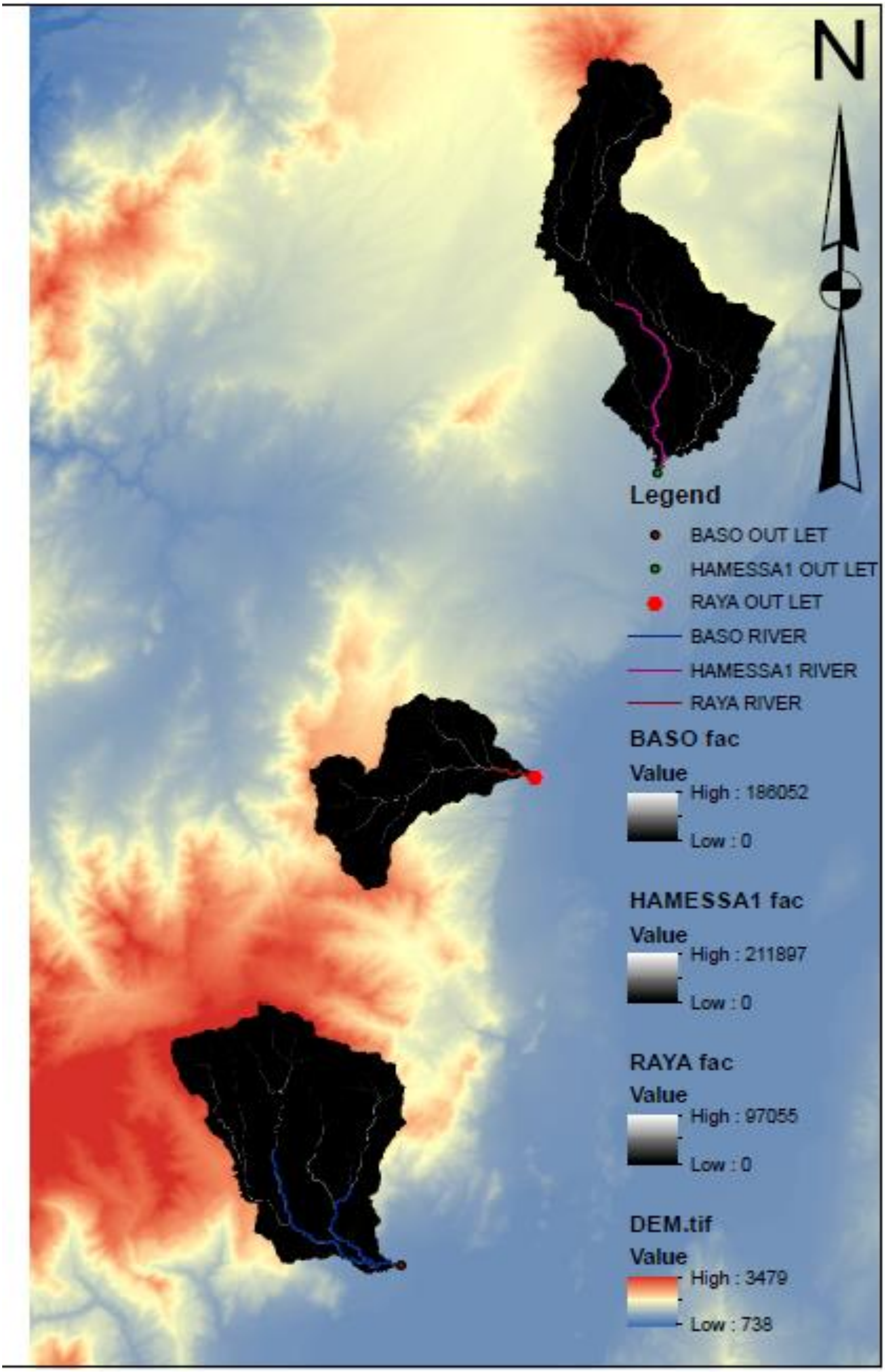
	Return Period, T [yr]	Exidence Probability	Skewness Coeff, Cs	*Frequency factor, K_T	Y_T	Rainfall X_T [m3/s]
Alaba Kulito	2.00	0.50	0.01	0.00	1.76	57.80
	5.00	0.20		0.84	1.85	70.43
	10.00	0.10		1.28	1.89	78.09
	25.00	0.04		1.75	1.94	87.19
	50.00	0.02		2.05	1.97	93.62
	100.00	0.01		2.33	2.00	99.80
Wolaita Sodo	2.00	0.50	0.07	-0.02	1.75	56.81
	5.00	0.20		0.84	1.84	69.78
	10.00	0.10		1.29	1.89	77.88
	25.00	0.04		1.78	1.94	87.70
	50.00	0.02		2.11	1.98	94.79
	100.00	0.01		2.40	2.01	101.72
Mirab Abaya	2.00	0.50	0.13	-0.02	1.80	62.65
	5.00	0.20		0.84	1.89	77.05
	10.00	0.10		1.29	1.93	86.05
	25.00	0.04		1.78	1.99	96.98
	50.00	0.02		2.11	2.02	104.86
	100.00	0.01		2.40	2.05	112.58
Arbaminch	2.00	0.50	0.29	-0.05	1.71	50.91
	5.00	0.20		0.82	1.80	63.43
	10.00	0.10		1.31	1.86	71.67
	25.00	0.04		1.85	1.91	82.11
	50.00	0.02		2.21	1.95	89.92
	100.00	0.01		2.54	1.99	97.79

* From standard frequency distribution table, f(Cs, T)

Appendix 1.2 - **Catchment area**

Catchm Area No.	*A1*	*A2*	*A3*	*A4*	*A5*
Area (km2)	*186.03*	*81.56*	*199.04*	*91.17*	*151.47*




N
Legend
BASO OUT LET
HAMESSA1 OUT LET
RAYA OUT LET
BASO RIVER
HAMESSA1 RIVER
RAYA RIVER
BASO fac
Value
High : 186052
Low : 0
HAMESSA1 fac
Value
High : 211897
Low : 0
RAYA fac
Value
High : 97055
Low : 0
DEM.tif
Value
High : 3479
Low : 738

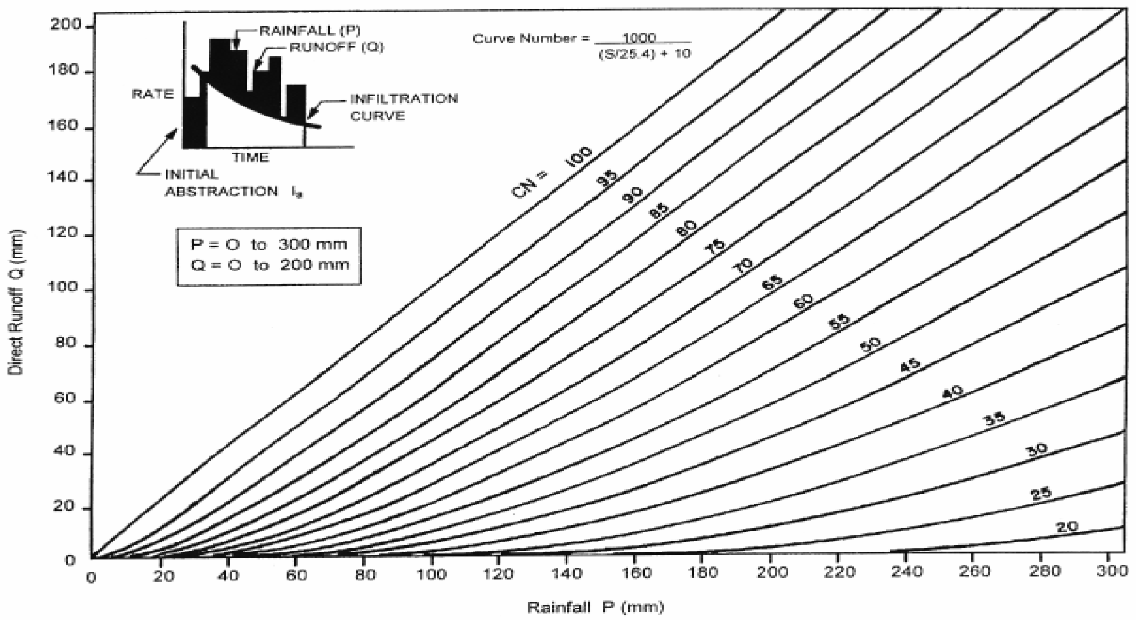
Appendix 1.3 - Curve Number (CN) Based on Soil Type

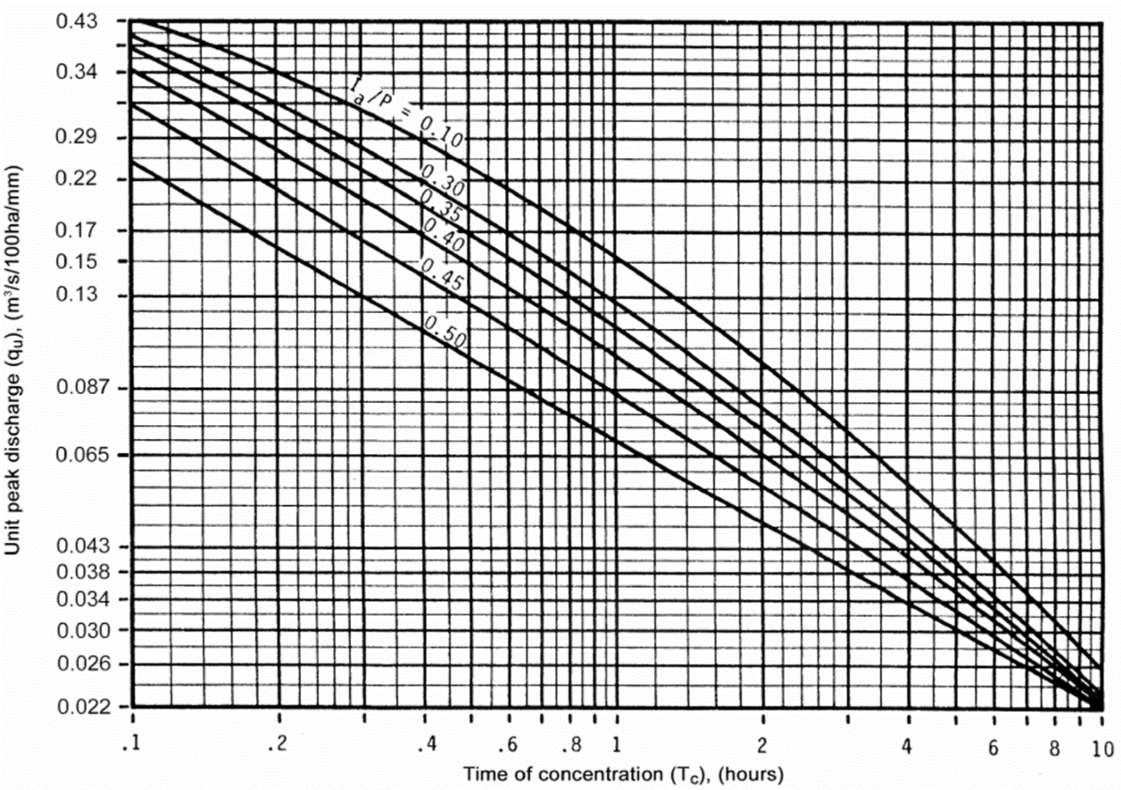
Catchment Area No.	Soil Group (%)		Weighted Average, CN
	Soil Type B	Soil Type D	
A1	100.0%	0.0%	64.2
A2	32.3%	67.7%	75.2
A3	19.4%	80.6%	75.5
A4	25.8%	74.2%	81.2
A5	98.4%	1.6%	73.2

Appendix 1.4 - Summary of Time of Concentration Calculation

Catchm Area No.	Stretch 1							
	Elev 1 (m)	Elev 2 (m)	Dist. (m)	El. Diff. (m)	Dist. (m)	Slope (m/m)	Tc (hr.)	
A1	1820	1690	11830	130	11.83	0.011	2.52	
A2	1940	1850	16180	90	16.18	0.006	4.17	
A3	1900	1350	26610	550	26.61	0.021	3.69	
A4	1700	1250	11010	450	11.01	0.041	1.44	
A5	1300	1150	5750	150	5.75	0.026	1.04	
Catchm Area No.	Stretch 2							
	Elev 1 (m)	Elev 2 (m)	Dist. (m)	El. Diff. (m)	Dist. (m)	Slope (m/m)	Tc (hr.)	
A1	2240	1820	14360	420	14.36	0.029	2.01	
A2	2065	1940	3860	125	3.86	0.032	0.7	
A3	2200	1900	6220	300	6.22	0.048	0.87	
A4	2150	1700	4800	450	4.8	0.094	0.55	
A5	2300	1300	6080	1000	6.08	0.164	0.53	
Catchm Area No.	Stretch 3							Total Tc
	Elev 1 (m)	Elev 2 (m)	Dist. (m)	El. Diff. (m)	Dist. (m)	Slope (m/m)	Tc (hr.)	
A1	2470	2240	1360	230	1.36	0.169	0.17	4.7
A2				0	0	0	0	4.87
A3	2950	2200	3130	750	3.13	0.24	0.28	4.84
A4				0	0	0	0	1.99
A5	2950	2300	12720	650	12.72	0.051	1.48	3.05

Appendix 1.5 - ERA DDM graph for Accumulated Direct R.O and Unit Peak Discharge





Appendix 1.6 - Summary of Discharge Calculation Based on SCS Method

Catch Area No.	River Name	Station	Area (km2)	Rainfall Depth, P (mm)	Time of Concent. Tc (hr)	Curve Number CN*	Rec. Int. (Year	Potential Maximum retention, Sr (mm)	Initial Abstruction , Ia (mm)	Accumulated Direct R.O Qd (mm)	Ia/P	Unit Peak Discharge, qu (m3/s)	Peak Discharge, qp (m3/s)
A1	Bishan Guracha	Alaba kulito	186.03	99.80	4.70	64.18	100	141.73	28.35	28.00	0.28	0.04	224.48
A1	Bishan Guracha	Alaba kulito	187.03	94.00	4.70	64.18	50	141.73	28.35	25.00	0.30	0.04	196.39
A1	Bishan Guracha	Alaba kulito	188.03	88.00	4.70	64.18	25	141.73	28.35	22.00	0.32	0.04	168.16
A2	Wedeba	Alaba kulito	81.56	99.80	4.87	75.17	100	83.92	16.78	46.00	0.17	0.05	176.33
A2	Wedeba	Alaba kulito	81.56	94.00	4.87	75.17	50	83.92	16.78	41.00	0.18	0.05	156.16
A2	Wedeba	Alaba kulito	81.56	88.00	4.87	75.17	25	83.92	16.78	36.50	0.19	0.05	138.13
A3	Hamessa 1	Wolaita Sodo	199.04	101.72	4.84	75.49	100	82.48	16.50	45.00	0.16	0.05	438.88
A3	Hamessa 1	Wolaita Sodo	199.04	94.79	4.84	75.49	50	82.48	16.50	40.00	0.17	0.05	382.15
A3	Hamessa 1	Wolaita Sodo	199.04	87.70	4.84	75.49	25	82.48	16.50	35.00	0.19	0.05	327.42
A4	Raya	Wolaita Sodo	91.17	101.72	1.99	81.16	100	58.95	11.79	56.00	0.12	0.10	488.58
A4	Raya	Wolaita Sodo	91.17	94.79	1.99	81.16	50	58.95	11.79	52.00	0.12	0.10	453.21
A4	Raya	Wolaita Sodo	91.17	87.70	1.99	81.16	25	58.95	11.79	47.00	0.13	0.10	408.34
A5	Baso	Arbaminch	151.47	97.79	3.05	73.18	100	93.10	18.62	40.08	0.19	0.07	394.62
A5	Baso	Arbaminch	151.47	89.92	3.05	73.18	50	93.10	18.62	36.00	0.21	0.06	351.05
A5	Baso	Arbaminch	151.47	82.11	3.05	73.18	25	93.10	18.62	30.00	0.23	0.06	281.74

Appendix 2 - Surveying Cross section Data's

Appendix 2.1 - Wedeba X-section data

x1	Y1	x2	Y2	x3	Y3	x4	Y4	x4.6	Y4.6
0	1840	0	1840	0	1843	0	1844.291	0	1844.539
1	1837.7	0.546	1839	4.403	1842.134	2.879	1844.11	1.887	1844.411
2	1836	2.0647	1838	5.656	1841.522	7.397	1842.874	7.165	1844.063
3	1836.3	3.5911	1837	5.927	1841.401	11.361	1842.231	11.933	1843.707
4	1837.2	4.9087	1836	6.615	1841.019	11.658	1842.277	12.119	1843.432
5	1838.1	5.0437	1836	7.029	1840.598	13.941	1842.813	12.643	1843.275
6	1839.1	5.8775	1837	7.155	1840.539	18.428	1839.704	19.258	1837.838
7	1839.3	7.0593	1838	8.121	1839.938	19.958	1837.777	20.041	1837.317
8	1839.5	8.24	1838.999	9.067	1839.337	21.786	1837.374	20.139	1837.299
9	1839.7	8.2411	1839	9.893	1839.198	27.426	1836.747	20.207	1837.298
10	1839.9	17.9633	1840	10.046	1836.547	28.93	1836.607	21.1	1837.392
11	1840.1			13.184	1836.547	29.463	1836.572	25.258	1837.364
				18.065	1836.547	29.751	1836.584	29.009	1837.499
				20.575	1836.547	31.408	1842.223	29.394	1842.412
				23.993	1840.234	35.895	1842.189	34.974	1841.67
				24.415	1840.397	36.268	1842.682	35.783	1842.268
				24.857	1840.484	37.747	1842.683	36.652	1842.289
				25.014	1840.585	38.005	1842.741	37.651	1842.273
				33.246	1841.446	48.671	1842.865	48.671	1843.019

x5	Y5	x6	Y6	x7	Y7	x8	Y8	x9	Y9
0	1840.512	0	1840	0	1840	0	1841	0	1841
5.294	1840.106	7.34	1839	2.67	1839	4.92	1840.006	2	1840.5
5.422	1840.086	7.52	1838.962	5.34	1838	4.952	1840	4	1839.5
6.156	1840.033	12.09	1838	5.68	1837.778	6.072	1839	6	1837.94
6.341	1839.931	14.84	1837.52	6.87	1837.72	7.192	1838	8	1837.94
8.568	1839.776	17.505	1837.52	8.3	1837.72	8.132	1837.82	10	1837.94
10.403	1839.349	20.825	1838	8.534	1837.72	8.912	1837.82	12	1838
14.596	1838.01	21.76	1838.204	10.452	1837.72	10.442	1837.82	14	1841
16.719	1837.411	25.412	1839	10.883	1838	11.442	1837.82		
21.179	1837.411	32.772	1840	11.343	1839	12.197	1838		
22.903	1837.411			13.503	1840	13.237	1839		
23.313	1837.861			13.69	1840.032	14.06	1839.281		
23.803	1838.389			19.403	1841	16.167	1840		
24.627	1839.317								
25.415	1839.748								
26.311	1839.776								
27.984	1840.113								
29.757	1840.578								
30.887	1840.766								

River Station	LOB	Channel	ROB	Left Bank Sta	Right Bank Sta
9	8.4197	12.6952	15.4933	2	14
8	9.04	10.1766	11.0654	4.92	14.06
7	23.9859	20.2091	18.602	2.67	13.69
6	19.3002	14.4005	12.7425	7.52	25.412
5	4.195	7.014	7.378	8.568	25.415
4.6	19.23	19.03	18.9	12.643	35.783
4.5	Bridge			Bridge	
4	9.271	9.169	9.236	13.941	35.895
3	30.721	25.4934	22.3208	9.893	25.014
2	12.6979	11.29	10.6157	0	8.24
1	0	0	0	0	9

Appendix 2.2 - Bishan Guracha X-section data

x1	Y1	x2	Y2	x3	Y3	x4.7	Y4.7	x7.4	Y7.4
0	1703	0	1702	0	1700	0	1701.75	0	1705.924
7.88	1702	6.617	1701	1.64	1699.64	6.36	1700.86	14.901	1705.263
15.95	1701	12.9	1700	4.56	1699	11.335	1700.066	15.395	1705.423
22.48	1700	19.366	1699	10.4	1698	12.297	1699.979	15.474	1705.438
27.62	1699.073	21.76	1698.446	17.19	1695.42	17.75	1699.859	15.722	1705.446
28.028	1699	23.688	1698	30.94	1695.42	20.135	1699.177	15.739	1705.178
33.419	1698	27.758	1695.02	31.18	1695.42	20.866	1699.001	16.357	1698.331
38.251	1694.8	33.944	1695.02	35.3	1698	22.811	1698.005	17.711	1698.168
44.391	1694.8	41.77	1695.02	42.26	1699	22.823	1696.02	26.747	1696.876
52.2	1694.8	42.4	1695.02	53.96	1700	27.978	1696.02	28.812	1696.517
56.414	1694.8	47.38	1697			33.423	1696.02	29.059	1696.623
60.304	1697	52.63	1698			39.859	1697.451	30.906	1704.606
65.021	1698	60.605	1699			42.005	1698.574	34.158	1705.316
72.659	1699	68.161	1700			42.207	1698.64	38.396	1705.26
79.625	1700	76.13	1701			46.126	1699.101		
						50.584	1699.658		
						53.178	1699.953		
						60.154	1700.59		
						72.645	1701.5		

x7.6	Y7.6	x9	Y9	x10	Y10
0	1706.165	0	1700	0	1700
15.365	1705.274	5.51	1699	2.76	1699
15.943	1705.56	9.4	1698.57	12.53	1698.309
16.35	1705.045	14.55	1698	16.89	1698
17.007	1698.36	25	1698	30.03	1698
27.133	1696.908	29.03	1698.552	36.77	1698.799
27.206	1696.897	32.3	1699	38.47	1699
28.809	1696.803	36.91	1700	42.58	1700
28.823	1696.804	41.39	1701	46.8	1701
30.776	1704.821	45.87	1702	51.13	1702
35.073	1705.174				
36.725	1705.166				
38.387	1705.131				
38.396	1705.131				

River Station	LOB	Channel	ROB	Left Bank Sta	Right Bank Sta
10	15.85	13.76	13.69	12.53	36.77
9	6.713	4.635	1.92	9.4	29.03
8.75*	6.713	4.635	1.92	5.725	34.008
8.5*	6.713	4.635	1.92	10.135	34.496
8.25*	14.312	12.655	11.42	10.502	38.516
7.6	19.32	19.32	19.32	16.35	30.776
7.5	Bridge			Bridge	
7.4	60.08	53.808	54.785	15.739	30.906
4.7	25.35	29.055	27.636	20.135	46.126
3	37.77	19.77	16.36	4.56	35.3
2	22.22	22.92	23.16	21.76	60.605
1	17.82	23.32	21.05	27.62	65.021

Appendix 2.3 - Hamessa-1 X-section data

x1	Y1	x2	Y2	x3	Y3	x4	Y4	x5	Y5
0	1373.96	0	1373	0	1374	0	1374	0	1374
9.287	1373.99	1.13	1372.51	3.65	1373.92	1.07	1373.51	0.14	1373.26
9.765	1373.89	1.19	1372	3.66	1374	1.1	1373.64	0.2	1372.45
10.454	1373.71	2.63	1371.57	3.79	1373.02	1.15	1373	0.25	1372
14.213	1373.81	2.64	1371.69	7.23	1372.37	3.57	1372.56	1.61	1371.62
15.065	1373.87	2.71	1371	7.26	1372	3.64	1372.26	1.69	1371
15.81	1370.77	3.78	1370.9	9.09	1371.62	3.65	1372.06	6.35	1370.557
18.057	1368	4.74	1370.81	9.13	1371	6.3	1371.15	7.26	1370.47
23.704	1368	4.8	1370.52	11.11	1370.801	6.31	1371	7.36	1370
27.602	1368	4.82	1370.7	12.31	1370.68	9.04	1370.629	11.65	1370.668
28.73	1368	4.87	1370	12.46	1370	9.77	1370.53	13.33	1370.93
29.743	1368	11.43	1370.547	19.58	1370.745	9.8	1370	13.34	1371
29.786	1368	12.31	1370.62	20.87	1370.88	15.79	1370.4	16.31	1371.74
43	1372.197	12.41	1370.87	20.88	1371	15.85	1371	16.34	1371.73
43.38	1372.169	12.42	1371	23	1371.8	16.51	1371.055	16.36	1372
47.094	1374.262	13.61	1371.37	23.02	1372	17.54	1371.14	18.99	1372.44
47.826	1374.014	13.69	1372	28.19	1372.9	17.63	1372	19.1	1373
49.671	1373.803	14.74	1372.72	28.21	1373	18.21	1372.67		
55.491	1373.813	14.76	1373			18.28	1372.66		
						18.32	1373		

x6	Y6	x7	Y7	x8	Y8	x9	Y9	x10	Y10
0	1374	0	1374	0	1374.296	0	1374.586	0	1374.797
0.8	1373.93	2.06	1373.35	5.519	1374.286	5.519	1374.576	5.519	1374.787
0.95	1373.71	2.15	1373	6.407	1374.498	6.407	1374.788	6.407	1374.999
1	1373	5.09	1372	7.928	1374.745	7.928	1375.036	7.928	1375.247
2.94	1372.85	6.32	1371.67	13.06	1372.653	13.06	1372.943	13.188	1373.069
3.06	1372	6.37	1371	13.188	1372.568	13.188	1372.858	14.363	1373.098
4.93	1371.39	6.89	1371.026	14.363	1372.597	14.363	1372.887	25.794	1374.323
4.96	1371	14.07	1371.387	25.794	1373.822	25.794	1374.112	27.414	1373.941
6.88	1370.802	18.32	1371.6	27.414	1373.44	27.414	1373.73	27.424	1373.922
7.19	1370.77	18.35	1372	27.424	1373.421	27.424	1373.711	27.577	1373.145
7.21	1370.63	27.74	1372.27	27.577	1372.644	27.577	1372.934	27.583	1373.144
7.3	1370	27.79	1372.25	28.451	1373.837	28.451	1374.127	28.451	1374.338
10.18	1370.07	27.91	1373	30.717	1371.083	30.717	1371.374	30.717	1371.585
10.27	1370.69			31.617	1370.405	31.617	1370.695	31.617	1370.906
10.31	1370.9			33.758	1370.365	33.758	1370.655	33.758	1370.866
10.32	1371			38.156	1370.198	38.156	1370.488	38.156	1370.699

10.5	1371.037			38.754	1370.219	38.754	1370.51	38.754	1370.721
12.23	1371.39			40.194	1370.312	40.194	1370.602	40.194	1370.813
12.3	1371.6			40.327	1371.17	40.327	1371.46	40.327	1371.671
12.34	1372			43.336	1374.266	43.336	1374.556	43.336	1374.767
15.87	1372.31			45.699	1374.205	45.699	1374.495	45.699	1374.706
15.94	1372.29			46.125	1374.104	46.125	1374.395	46.125	1374.606
15.95	1372.1			46.585	1374.285	46.585	1374.575	46.585	1374.786
16.08	1373			48.867	1374.385	48.867	1374.676	48.867	1374.887
				55.491	1374.358	55.491	1374.648	55.491	1374.859

x11	**Y11**	**x12**	**Y12**	**x13**	**Y13**	**x14**	**Y14**	**x15**	**Y15**
0	1375	0	1374	0	1374	0	1374	0	1374
5.82	1374.991	3.93	1373.615	1.85	1373.29	2.15	1373.48	0.4	1373.85
7.602	1375.014	4.029	1372.995	1.88	1373	2.18	1373	0.42	1373.75
7.665	1375.014	5.82	1372.693	2.74	1372.08	2.56	1372.09	0.47	1373
8.397	1374.838	5.969	1372.667	2.79	1372.29	2.57	1372	0.81	1372.28
24.91	1372.668	7.531	1372.508	2.81	1372	3.01	1371.2	0.83	1372
25.705	1372.607	7.574	1372.309	3.55	1371.63	3.03	1371	1.67	1371
25.748	1373.067	7.659	1371.868	3.77	1371	4.21	1371.121	3.48	1371.093
27.888	1373.346	8.008	1371.613	4.39	1371.008	10.88	1371.803	10.43	1371.452
31.787	1371.12	8.84	1371.286	13.08	1371.116	11.93	1371.91	11.94	1371.53
37.434	1371.034	8.874	1371.001	13.41	1371.12	11.94	1372	11.98	1372
40.425	1373.697	18.413	1371.159	13.47	1372	16.49	1372.78	18.14	1372.8
41.277	1374.724	18.85	1371.387	19.02	1372.58	16.52	1373	18.17	1373
45.037	1374.63	19.261	1371.402	19.05	1373	25.43	1373.75	25.19	1373.73
45.725	1375.176	19.319	1372.011	26.97	1373.19	25.48	1374	25.2	1374
46.203	1375.071	21.915	1372.229	27.09	1373				
47.921	1375.087	21.944	1372.822	27.19	1373.92				
55.491	1375.063	25.083	1373.036	27.21	1373.93				
		34.012	1373.591	27.22	1374				
		34.292	1374						

x16	Y16	x17	Y17	x18	Y18	x19	Y19	x20	Y20
0	1374	0	1374	0	1374	0	1374	0	1377.203
7.8	1373.26	0.65	1373.12	0.85	1373.88	2.88	1373.54	5.82	1377.194
9.53	1373.08	0.66	1373	0.87	1374	3.01	1373	7.602	1377.217
9.8	1373.46	1.08	1372.51	0.96	1373.56	5.55	1372.58	7.665	1377.217
10	1373	1.12	1372.71	1.09	1373.26	5.58	1372	8.397	1377.041
10.46	1373.95	1.15	1372	1.15	1373.4	5.8	1372.01	12.154	1376.547
10.94	1373.42	5.17	1371.62	1.24	1373.38	17.08	1372.515	24.91	1374.871
11.22	1374	5.2	1371	1.4	1373	19.87	1372.64	25.705	1374.81
15.68	1373	12.71	1371.03	3.91	1372.34	19.91	1373	25.748	1375.27
18.05	1372.3	12.78	1372	3.95	1372	25.88	1373.28	27.888	1375.549
19.96	1371.69	17.66	1372.12	6.3	1372.133	25.96	1374	31.787	1373.323
20.02	1371	17.79	1373	18.66	1372.835			37.434	1373.237
28.93	1371.23	19.52	1373.12	20.34	1372.93			40.425	1375.9
29.04	1371.88	19.58	1374	22.39	1373.54			41.277	1376.927
29.05	1372	20.51	1374	22.41	1374			45.037	1376.833
32.77	1372.95								
36.93	1373.18								
37	1373.05								
37.06	1374								

River Station	LOB	Channel	ROB	Left Bank Sta	Right Bank Sta
20	19.2	19.07	19.15	7.665	41.277
19	11.42	11.35	11.39	5.8	17.08
18	9.35	7.85	6.73	6.3	18.66
17	11.44	11.25	10.93	0	20.51
16	17.13	16.8	16.95	19.96	28.96
15	3.32	4.53	6.98	3.48	10.43
14	10.92	10.82	10.68	4.21	10.88
13	9.35	8.43	7.34	4.39	13.08
12.2	10.343	8.605	6.45	9.325	18.413
11	20.5	20.5	20.5	7.665	41.277
10.5	Bridge			Bridge	
10	9.55	9.7	9.979	7.928	43.336
9	13.13	13.337	13.72	7.928	43.336
8	14.14	14.17	14.21	7.928	43.336
7	18.8	18.79	18.09	2.06	27.79
6	17.92	9.04	4.58	2.94	15.87
5	16.57	12.7	7.09	6.35	11.65
4	14.21	15.69	16.99	3.57	17.63
3	9.56	13.03	15.37	3.65	28.21
2	14.65	18.25	22.71	3.78	14.76
1	4.96	9.36	6.91	10.454	47.094

Appendix 2.4- Raya X-section data

X1	Y1	X3	Y3	X5	Y5	X5.5	Y5.5	X6
0	1215	0	1215	0	1216.333	0	1216.667	0
0.181	1214	4.0321	1214	3.357	1215.996	4.293	1216.302	15.022
3.9366	1214	6.1042	1213	3.426	1215.533	9.629	1215.709	17.023
7.0942	1213	6.1142	1213	7.528	1215.298	9.637	1215.629	25.242
14.5871	1213	16.3159	1213	7.535	1215.258	10.951	1215.02	28.005
26.4575	1214	40.5936	1214	8.562	1214.939	12.312	1214.513	28.624
26.458	1215	40.8794	1215	9.626	1214.671	12.363	1214.516	35.465
X2	**Y2**	**X4**	**Y4**	9.666	1214.701	12.396	1214.481	37.938
0	1215	0	1216	9.691	1214.703	12.453	1214.379	40.289
2.88	1214.31	2.42	1215.69	9.704	1214.629	12.47	1214.263	41.395
2.91	1214.23	2.47	1215	9.723	1214.672	12.614	1214.001	43.738
2.94	1214.16	6.94	1214.83	9.749	1214.525	13.638	1214.008	46.811
2.96	1214	7.01	1214.97	9.862	1214	13.689	1214.008	49.26
9.15	1213.21	7.11	1214	10.663	1214.005	35.282	1214.148	49.672
9.2	1213	7.6873	1214.003	10.688	1214.005	35.296	1214.001	50.422
9.24	1213.08	7.84	1214.004	21.485	1214.075	35.358	1214.029	50.693
25.85	1213.56	45.91	1214.21	21.492	1214.002	35.579	1214.129	51.236
25.87	1213.74	45.98	1215	22.099	1214.339	35.65	1214.669	59
25.88	1214	47.6392	1215	48.452	1214.505	50.994	1214.799	61.107
32.3485	1214.042	47.97	1215	48.501	1215.031	51.023	1215.063	
35.15	1214.06			49.672	1215.033	51.704	1215.065	
35.25	1215			49.688	1215.033	51.734	1215.066	
35.32	1215			51.933	1215.077	55.902	1215.153	
				51.97	1215.333	55.97	1215.667	

Y6	X7	Y7	X7.5	Y7.5	X8	Y8
1219.035	0	1218.744	0	1217.872	0	1217
1219.122	10.336	1218.614	3.495	1217.308	4.05	1216
1215.172	14.529	1218.633	8.729	1217.179	10.1146	1215.757
1214.208	16.295	1215.245	10.528	1217.174	10.72	1215.733
1214.178	23.549	1214.411	13.048	1217.166	15.55	1215.54
1213.97	25.988	1214.382	14.11	1215.465	15.6	1215.58
1214.115	26.534	1214.203	18.471	1215.018	15.66	1215
1213.865	32.273	1214.315	20.266	1214.903	33.7	1215.24
1214.133	34.212	1214.158	23.716	1214.935	33.75	1215
1214.569	34.459	1214.139	24.881	1214.849	33.85	1215.82
1214.274	34.755	1214.027	25.029	1214.859	33.87	1216
1214.516	37.333	1214.061	25.208	1214.514	36.8968	1216.242

1215.666	37.34	1214.027	35.516	1214.651	37.36	1216.279
1216.923	37.701	1214.184	36.884	1215.221	46.26	1216.99
1218.215	39.438	1214.405	37.514	1215.42	53.83	1217.36
1219.105	40.425	1214.782	38.848	1215.32	53.92	1218
1219.149	42.516	1214.536	40.598	1215.458	**X9**	**Y9**
1219.032	45.258	1214.752	41.993	1215.979	0	1218
1219	47.443	1215.744	42.809	1217.469	3.7377	1218
	47.811	1216.823	43.196	1217.5	6.5415	1217
	48.722	1218.696	43.549	1217.528	9.0546	1216
	49.031	1218.721	50.615	1217.882	9.8701	1216
	54.969	1218.774	56.925	1218.063	50.0705	1216
	60.08	1218.857	57	1218.429	52.1817	1216
					81.7246	1217

River Station	LOB	Channel	ROB	Left Bank Sta	Right Bank Sta
11	19.8023	30.7234	45.6945	7.0827	43.4092
10	17.08	30.6	47.23	13.9242	48.2837
9	23.7526	22.0994	22.9215	9.8701	50.0705
8	13.353	10.137	7.173	10.1146	36.8968
7.5	13.353	10.137	7.173	8.729	42.809
7	22.2357	21.1637	20.1757	7.343	48.722
6.5	Bridge			Bridge	
6	29.39	36.167	42.11	15.022	59
5.5	9.64	7.777	4.05	13.638	51.704
5	9.64	7.777	4.05	10.663	49.672
4	25.66	25.52	25.63	7.6873	47.6392
3	14.9781	15.7643	7.9276	6.1142	40.5936
2	21.3	21.38	22.14	0	32.3485
1	15.08	16.85	17.84	0	26.4575

Appendix 2.5 - Baso X-section data

x1	Y1	x3	Y3	x4.5	Y4.5	x5	Y5
0	1189	0	1189	0	1188.999	0	1188.499
11.4	1188	2.92	1190	8.753	1188.853	10.667	1188.811
19.13	1187	5.03	1191	9.747	1188.874	11.131	1188.232
19.966	1186	9.9	1191	10.171	1188.488	11.657	1188.105
20.98	1186	11.21	1190	10.651	1188.403	12.162	1187.824
38.806	1186	12.22	1189	11.113	1188.216	20.281	1187.442
40.116	1187	13.67	1188	13.055	1188.149	25	1186.694
45.05	1187.748	15.25	1187	18.531	1187.825	25.44	1186.624
46.716	1188	16.76	1187	22.843	1187.22	25.712	1186.581
55.776	1188	16.82	1186.995	23.168	1187.17	27.927	1186.653
x2	**Y2**	29.29	1186	23.369	1187.14	27.983	1186.585
0	1189	50.59	1186	25.005	1187.173	45.814	1186.704
12.32	1189	51.28	1186	25.047	1187.127	46.078	1186.573
22.31	1189	57.26	1187	32.937	1187.104	46.364	1186.687
22.96	1188	64.66	1188	35.675	1186.787	53.546	1187.577
23.32	1187	70.09	1189	38.217	1186.493	61.137	1187.782
23.93	1186.927	75.37	1189	38.412	1186.382	61.884	1188.11
31.72	1186	**x4**	**Y4**	39.215	1186.382	64.176	1188.573
51.41	1186	0	1190	39.517	1186.461	64.33	1188.394
52.36	1186	7.1	1189	47.091	1187.141	64.344	1188.579
53.57	1187	10.59	1189	55.096	1187.37	64.928	1188.182
58.03	1188	18.53	1188.271	55.884	1187.597	73	1188.197
61.71	1188	21.48	1188	58.301	1187.934	74.52	1188.2
70.43	1187	22.28	1187	58.463	1187.816		
		23.08	1186	58.478	1187.94		
		25.49	1186	59.094	1187.682		
		56.82	1186.974	67.607	1187.789		
		57.64	1187	67.927	1187.798		
		58.96	1188	68.442	1188.132		
		61.09	1189	69.273	1188.465		
		65.96	1189	71.172	1188.466		
		71.31	1189	73.26	1188.466		
		74.66	1190	74.567	1188.8		

x6	Y6	x6.3	Y6.3	x7	Y7	x9	Y9
0	1188.266	0	1187.844	0	1187	0	1189
9.448	1188.594	9.214	1188	14.14	1187	6.41	1188
11.175	1188.551	12.895	1188.315	21.61	1188	11.86	1187.71
11.914	1187.965	14.082	1188.382	24.93	1189	25.19	1187
14.484	1187.418	15.252	1188.548	35.89	1188	30.09	1187
19.796	1187.215	16.245	1188.316	42.73	1187	41.73	1187.869
24.178	1188.172	16.26	1188.309	49.88	1187	43.49	1188
24.47	1188.042	19.768	1187.781	51.25	1187	46.6	1188
25.84	1187.432	23.387	1187.544	69.13	1187	54.07	1187
27.426	1186.726	27.018	1187.205	83.33	1187	**x10**	**Y10**
45.767	1186.742	27.845	1187.232	89.647	1187.738	0	1188
53.317	1186.661	32.504	1187.729	91.89	1188	1.78	1189
54.892	1186.651	32.998	1187.781	94.31	1189	2.35	1189
61.968	1187.317	33.397	1187.695	96.41	1189	4.46	1188
63.225	1186.751	34.31	1187.288	99.01	1188	10	1187.623
64.236	1188.598	35.367	1186.817	101.67	1187	23.55	1186.7
64.688	1188.238	47.595	1186.828	**x8**	**Y8**	34.14	1186.867
65.525	1188.734	52.628	1186.774	0	1187	42.55	1187
65.53	1188.734	53.678	1186.767	4.54	1188	**x11**	**Y11**
70.33	1188.601	59.638	1186.767	9.9	1189	0	1188
74.4	1188.489	64.371	1186.767	15.03	1190	7.38	1188
		70.489	1187.375	20.18	1190	9.78	1187.829
		71.576	1187.027	25.89	1189	21.38	1187
		72.45	1188.281	33.65	1188	27.1	1187.393
		72.841	1188.052	37.73	1187.694	35.95	1188
		73.565	1188.402	46.98	1187		
		75.42	1188.459	64.36	1187		
		77.417	1188.759	72.15	1187.674		
		78.938	1188.734	75.91	1188		
		79.15	1188.73	84.49	1188		
		81.295	1188.362	90.69	1187		
		83.49	1187.993				

River Station	LOB	Channel	ROB	Left Bank Sta	Right Bank Sta
11	57.57	54.67	52.39	9.78	27.1
10	19.55	18.88	18.85	10	34.14
9	15.24	15.53	14.49	11.86	41.73
8	16.94	16.25	16.53	37.73	72.15
7	6.124	8.39	9.036	51.25	89.647
6.3	4.062	5.195	5.518	33.397	73.565
6	17.65	17.65	17.65	24.47	65.53
5.5	Bridge			Bridge	
5	3.365	5.505	6.735	25	73
4.5	5.93	10.21	12.67	22.843	67.607
4	12.19	11.51	11.58	18.53	56.82
3	21.65	20.31	20.33	16.82	50.59
2	11.87	9.63	6.07	23.93	51.41
1	21.83	15.87	10.74	20.98	45.05

Appendix 3 - Values of Roughness Coefficient n (Uniform Flow)

EXCAVATED OR DREDGED

Type of Channel and Description	Minimum	Normal	Maximum
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.02
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.03
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.03
2. Grass, some weeds	0.025	0.03	0.033
3. Dense Weeds or aquatic plants in deep channels	0.03	0.035	0.04
4. Earth bottom and rubble sides	0.025	0.03	0.035
5. Stony bottom and weedy sides	0.025	0.035	0.045

6. Cobble bottom and clean sides	0.03	0.04	0.05

c. Backhoe-excavated or dredged

1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.05	0.06

d. Rock cuts

1. Smooth and uniform	0.025	0.035	0.04
2. Jagged and irregular	0.035	0.04	0.05

e. Channels not maintained, weeds and brush uncut

1. Dense weeds, high as flow depth	0.05	0.08	0.12
2. Clean bottom, brush on sides	0.04	0.05	0.08
3. Same, highest stage of flow	0.045	0.07	0.11
4. Dense brush, high stage	0.08	0.1	0.14

NATURAL STREAMS

Type of Channel and Description	Minimum	Normal	Maximum

1. Minor streams (top width at flood stage < 30 m)

a. Streams on Plain

1. Clean, straight, full stage, no rims or deep pools	0.025	0.03	0.033
2. Same as above, but more stones and weeds	0.03	0.035	0.04
3. Clean, winding, some pools and shoals	0.033	0.04	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.05
5. Same as above, lower stages, more ineffective slopes and sections	0.04	0.048	0.055
6. Same as 4, but more stones	0.045	0.05	0.06
7. Sluggish reaches, weedy, deep pools	0.05	0.07	0.08
8 Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.1	0.15

b. Mountain streams, no vegetation in channel, banksusually steep, trees and brush along banks submerged at high stages

1. Bottom: gravel, cobbles, and few boulders	0.03	0.04	0.05
2. Bottom: cobbles with large boulders	0.04	0.05	0.07

2. Flood Plains

a. Pasture, no brush

1. Short grass	0.025	0.03	0.035
2. High grass	0.03	0.035	0.05

b. Cultivated area

1. No crop	0.02	0.03	0.04
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.03	0.04	0.05

c. Brush

1. Scattered brush, heavy weeds	0.035	0.05	0.07
2. Light brush and trees in winter	0.035	0.05	0.06
3. Light brush and trees, in summer	0.04	0.06	0.08
4. Medium to dense brush, in winter	0.045	0.07	0.11
5. Medium to dense brush, in summer	0.07	0.1	0.16

d. Trees

1. Dense willows, summer, straight	0.11	0.15	0.2
2. Cleared land with tree stumps, no sprouts	0.03	0.04	0.05
3. Same as above, but with heavy growth of spouts	0.05	0.06	0.08
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.08	0.1	0.12
5. Same as above, but with flood stage reaching branches	0.1	0.12	0.16

3. Major Streams (top width at flood stage > 30 m).similar description, because banks offer less The n value is less than that for minor streams of similar description

a. Regular section with no boulders or brush	0.025	--	0.06
b. Irregular and rough section	0.035	--	0.1

4 Various Open Channel Surfaces

a. Concrete	0.012-	0.02	

b. Gravel bottom with:

1. Concrete	0.02		
2. Mortared stone	0.023		
3. Riprap	0.033		

c. Natural Stream Channels

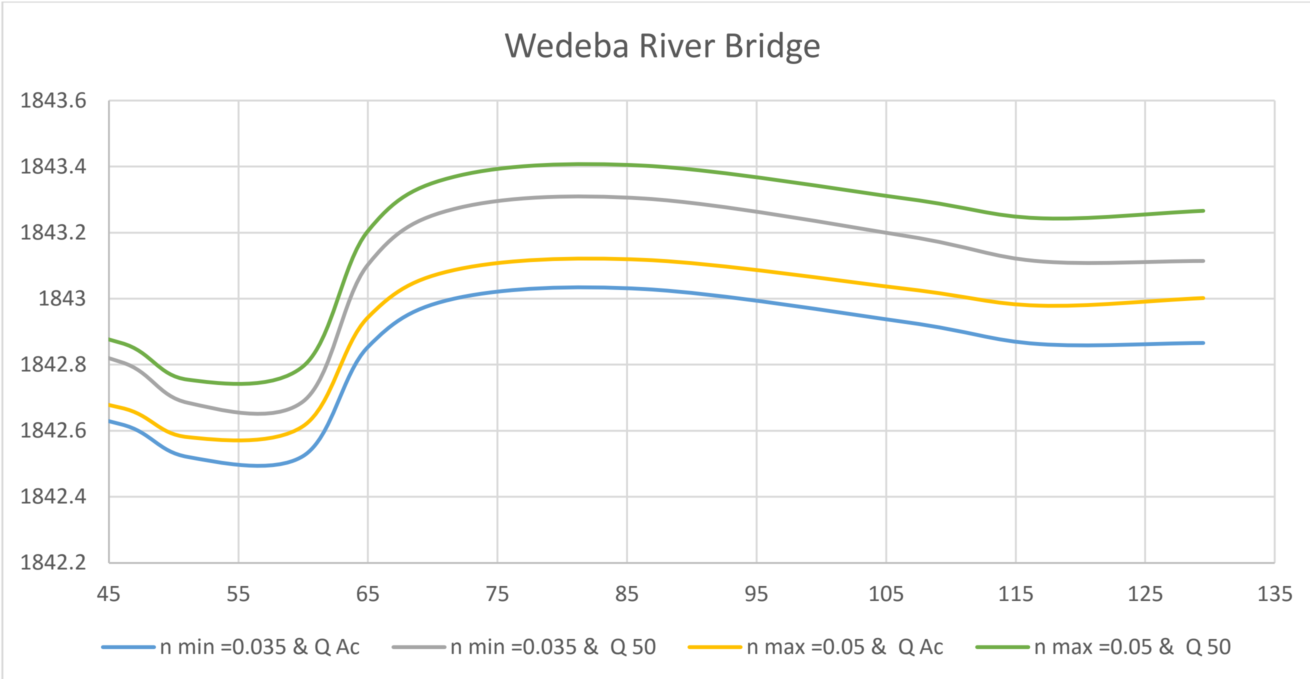
1. Clean, straight stream	0.03		
2. Clean, winding stream	0.04		
3. Winding with weeds and pools	0.05		

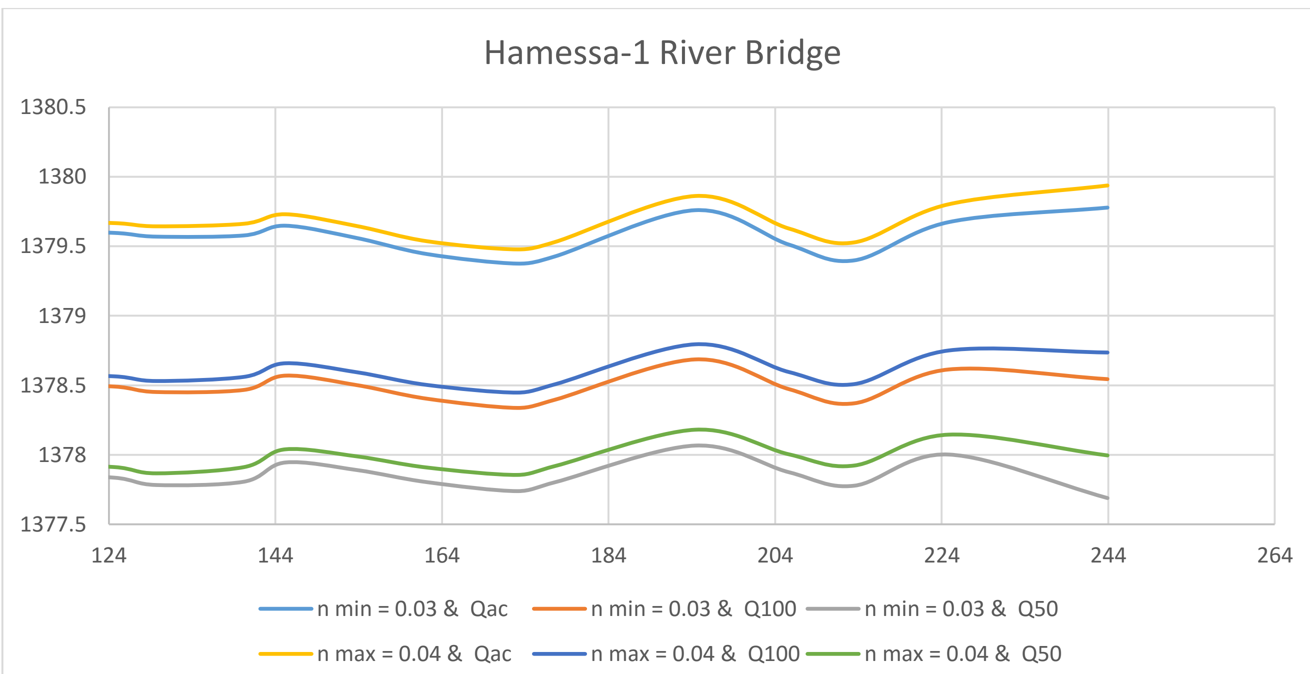
4. With heavy brush and timber	0.1		

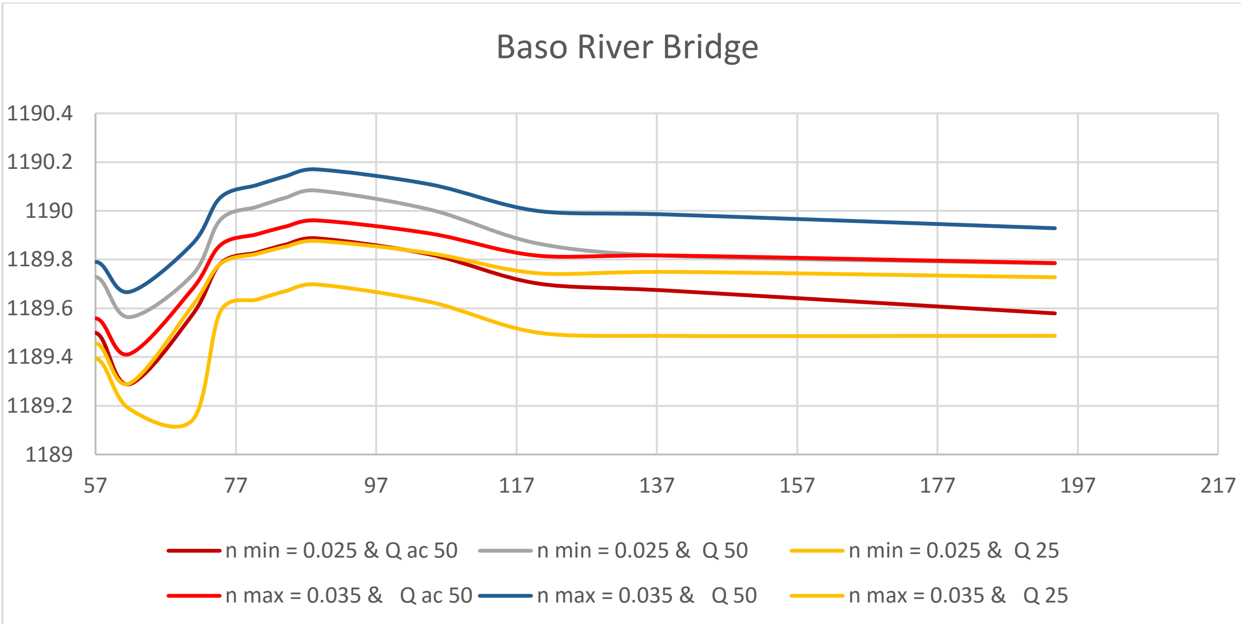
d. Flood Plains

1. Pasture	0.035		
2.Field Crops	0.04		
3. Light Brush and Weeds	0.05		
4. Dense Brush	0.07		
5. Dense Trees	0.1		

Appendix 4 – manning Sensitivity






Baso River Bridge
1190.4
1190.2
1190
1189.8
1189.6
1189.4
1189.2
1189
57
77
97
117
137
157
177
197
217
n min = 0.025 & Q ac 50
n min = 0.025 & Q 50
n min = 0.025 & Q 25
n max = 0.035 &   Q ac 50
n max = 0.035 &   Q 50
n max = 0.035 &   Q 25